VOLUME 84 NO. SM2
MAY 1958
PART 1

JOURNAL of the

Soil Mechanics and Foundations Division

PROCEEDINGS OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

BASIC REQUIREMENTS FOR MANUSCRIPTS

This Journal represents an effort by the Society to deliver information to the reader with the greatest possible speed. To this end the material herein has none of the usual editing required in more formal publications.

Original papers and discussions of current papers should be submitted to the Manager of Technical Publications, ASCE. The final date on which a discussion should reach the Society is given as a footnote with each paper. Those who are planning to submit material will expedite the review and publication procedures by complying with the following basic requirements:

1. Titles should have a length not exceeding 50 characters and spaces.

2. A 50-word summary should accompany the paper.

3. The manuscript (a ribbon copy and two copies) should be double-spaced on one side of 8½-in. by 11-in. paper. Papers that were originally prepared for oral presentation must be rewritten into the third person before being submitted.

4. The author's full name, Society membership grade, and footnote reference stating present employment should appear on the first page of the paper.

5. Mathematics are reproduced directly from the copy that is submitted. Because of this, it is necessary that capital letters be drawn, in black ink, ⅛-in. high (with all other symbols and characters in the proportions dictated by standard drafting practice) and that no line of mathematics be longer than 6½-in. Ribbon copies of typed equations may be used but they will be proportionately smaller in the printed version.

6. Tables should be typed (ribbon copies) on one side of 8½-in. by 11-in. paper within a 6½-in. by 10½-in. invisible frame. Small tables should be grouped within this frame. Specific reference and explanation should be made in the text for each table.

7. Illustrations should be drawn in black ink on one side of 8½-in. by 11 in. paper within an invisible frame that measures 6½-in. by 10½-in.; the caption should also be included within the frame. Because illustrations will be reduced to 69% of the original size, the capital letters should be ⅛-in. high. Photographs should be submitted as glossy prints in a size that is less than 6½-in. by 10½-in. Explanations and descriptions should be made within the text for each illustration.

8. Papers should average about 12,000 words in length and should be no longer than 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

Further information concerning the preparation of technical papers is contained in the "Technical Publications Handbook" which can be obtained from the Society.

Reprints from this Journal may be made on condition that the full title of the paper, name of author, page reference (or paper number), and date of publication by the Society are given. The Society is not responsible for any statement made or opinion expressed in its publications.

This Journal is published quarterly by the American Society of Civil Engineers. Publication office is at 2500 South State Street, Ann Arbor, Michigan. Editorial and General Offices are at 33 West 39 Street, New York 18, New York. $4.00 of a member's dues are applied as a subscription to this Journal. Second-class mail privileges are authorized at Ann Arbor, Michigan.

Journal of the

SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

SOIL MECHANICS AND FOUNDATIONS DIVISION
EXECUTIVE COMMITTEE
Ralph E. Fadum, Chairman; Stanley J. Johnson, Vice-Chairman;
Thomas M. Leps; Ralph B. Peck; Jorj O. Osterberg, Secretary

COMMITTEE ON PUBLICATIONS
H. Bolton Seed, Chairman; John A. Focht, Jr.; Harold J. Gibbs;
James P. Gould; Gerald A. Leonards; Ralph B. Peck; Frank E. Richart, Jr.;
Woodland G. Shockley; Thomas H. Thornburn

CONTENTS

May, 1958

Papers

Number

Increased Resistance to Deformation of Clay Caused by Repeated Loading
by H. B. Seed, R. L. McNeill, and J. de Guenin 1645

Geology and Soils of the Newark (N. J.) Metropolitan Area
by Alfreds R. Jumikis 1646

Development of Multiple-Wheel CBR Design Criteria
by C. R. Foster and R. G. Ahlvin 1647

Geological Factors in Tunnel Construction: Tools and Techniques
by Arthur B. Cleaves 1648

Geological Factors in Tunnel Construction: Geology and Tunnel Design
A. B. Reeves 1649

Geological Factors in Tunnel Construction: Geophysical Investigations for the Lehigh Tunnel
by H. LeRoy Scharon and Arthur B. Cleaves 1650

(Over)

Copyright 1958 by the American Society of Civil Engineers.

Number

The Structure of Compacted Clay
by T. William Lambe 1654

The Engineering Behavior of Compacted Clay
by T. William Lambe 1655

Discussion 1657

Paper 1645 May, 1958 SM 2

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

INCREASED RESISTANCE TO DEFORMATION OF CLAY CAUSED BY REPEATED LOADING

H. B. Seed,[1] A. M. ASCE, R. L. McNeill,[2] J. M. ASCE and J. de Guenin[3]
(Proc. Paper 1645)

SYNOPSIS

The influence of stress history, in the form of a series of repeated stress applications, in increasing the resistance to deformation of compacted specimens of silty clay is described and possible causes of the effect are presented. The significance of the results in assessing the design life of highway pavements is discussed.

Stress history has been shown to have significant effects on various aspects of soil behavior. It has long been recognized that stress history has an important effect in determining the consolidation and strength characteristics of saturated clays and more recently it has been shown that changes in the sequence of pressure application can also affect the swelling characteristics of clays. It is perhaps not altogether surprising, therefore, to find that stress history, in the form of a series of applications of a constant stress, affects the deformation characteristics of partially saturated clays. Nevertheless, it might not have been anticipated that repeated loading would increase rather than decrease the resistance to deformation of soils having an initially high degree of saturation.

In recent years the Soil Mechanics Laboratory of the University of California has been conducting a study of soil deformation under repeated loading, wherein soil specimens in triaxial compression cells are subjected to a series of applications of a constant axial stress.(1) The primary purpose of these

Note: Discussion open until October 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1645 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 2, May, 1958.

1. Associate Prof. of Civ. Eng., Univ. of California, Berkeley, Calif.
2. Engr., Woodward, Clyde and Associates, Oakland, Calif.
3. Formerly Graduate Research Engr., Inst. of Transportation and Traffic Eng., Univ. of California, Berkeley, Calif.

investigations is to determine the deformation occurring during the period of application of repeated loads, many of which are too light to cause failure. However, when the necessary information on this aspect of soil behavior has been obtained, the specimens are usually loaded to failure as in the normal type of strength test. It was observed on a number of occasions that specimens loaded to failure in this way appeared to be appreciably stronger than comparable specimens not previously subjected to repeated loading and, in a general way, greater strength increases resulted from increased intensities of the repeated stress applications.

A typical example of this behavior is shown in Fig. 1 which presents the results of tests on four specimens trimmed from the same compacted sample of silty clay soil. The specimens were first subjected to a confining pressure of 1 kg. per sq. cm. in triaxial compression cells. Two of the specimens were then subjected to 12,000 and 60,000 repetitions of a deviator stress of 1.8 kg. per sq. cm. causing axial strains of 0.55% and 0.8% respectively. The other two specimens were subjected to 10,000 and 65,000 repetitions of a deviator stress of 3.6 kg. per sq. cm. causing axial strains of 1.45%and 2.12% respectively. The four specimens were then loaded to failure. The specimen with the least deformation during repeated loading had a strength of 5.4 kg. per sq. cm. and the specimen with the highest deformation during repeated loading had a strength of 8.75 kg. per sq. cm. The specimens with 0.8% and 1.45% strains during repeated loading had strengths of 5.9 and 7.5 kg. per sq. cm. respectively. Thus in this case the repeated application of a constant stress caused an increase in strength of more than 50% for the strongest specimen.

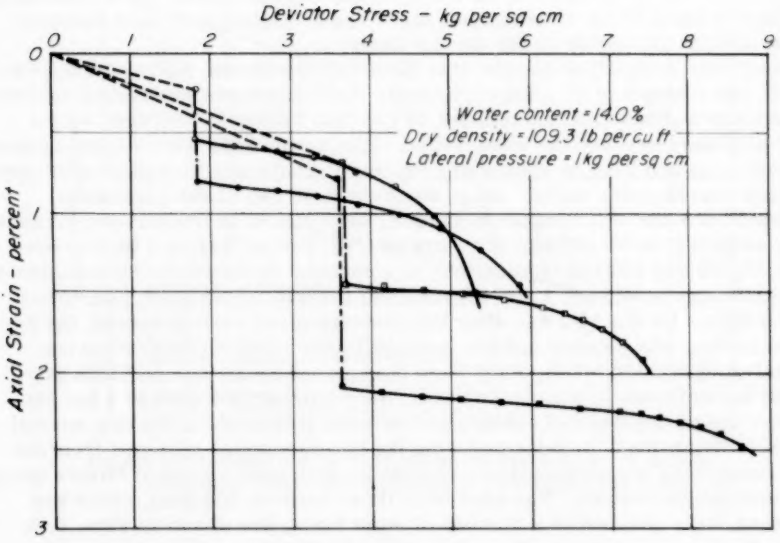
Since the tests described above were conducted on partially saturated specimens, the increase in strength due to repeated load applications may be attributed partly to an increase in density of the specimens during the tests; a part of the measured strength increase may also be due to an increase in thixotropic strength with time.(2) However, the magnitude of the strength increase is rather surprising in view of the relatively small deformation occurring during repeated loading and the short time interval of several days between the various specimens being loaded to failure.

It is also interesting that an increase in density could possibly have occurred in these tests. The test results shown in Fig. 1 were obtained on samples compacted by 125 tamps, using a tamping pressure of 225 psi to a relative density of 90%, based on the modified AASHO Compaction Test. If, as the strength data might imply, a series of applications of the 1.8 kg. per sq. cm. deviator stress (corresponding to a total axial stress of 40 psi) can cause a further increase in density, the beneficial effects of traffic loads in increasing the density of partially saturated subgrades might have significant effects in the performance of highway pavements.

In view of this possible significance, it was thought desirable to investigate further the magnitude of the strength increase which might result from repeated stress applications, and the influence of densification on this effect.

Effect of Repeated Loading on Soil Strength

In order to study the effect of repeated loading on the strength of soil specimens, a comprehensive series of tests were conducted on a silty clay soil from Vicksburg, Mississippi. This soil had a Liquid Limit of 37 and a Plastic Limit of 23.



- ● Loaded to failure in approx. 5 min after 12,000 cycles with deviator stress 1.8 kg per sq cm
- ○ Loaded to failure in approx. 5 min after 60,000 cycles with deviator stress 1.8 kg per sq cm
- ■ Loaded to failure in approx. 5 min after 10,000 cycles with deviator stress 3.6 kg per sq cm
- □ Loaded to failure in approx. 5 min after 65,000 cycles with deviator stress 3.6 kg per sq cm

----Stress vs strain for first application of repeated load.

——Deformation during repeated loading.

Fig. 1 - EFFECT OF REPEATED LOADING ON THE STRENGTH OF PARTIALLY SATURATED SILTY CLAY.

Tests were performed on samples of various water contents having degrees of compaction varying from 90 to 95%, based on the modified AASHO Compaction Test. The soil was mixed to the desired water content one day before compaction and stored for 24 hours in an airtight container. Samples were then compacted in 6" diameter molds, using the Triaxial Institute Kneading Compactor(5) to form specimens 4-1/2" high. All specimens were compacted in 5 layers using 15 or 25 tamps per layer and a tamping pressure between 75 and 350 psi depending on the density desired.

From each compacted sample four identical specimens, having a diameter of 1.4" and a height of 4" , were trimmed. Each specimen was placed between a lucite cap and base and surrounded by two thin rubber membranes with a layer of grease between the membranes. The membranes were sealed against the lucite cap and base by means of neoprene O-rings and the entire specimen was then stored under water. After about ten days two of the specimens trimmed from any one compacted sample, were placed in triaxial compression cells, subjected to a confining pressure of 1 kg. per sq. cm. and then to between 80,000 and 180,000 applications of a constant deviator stress sufficiently large to cause between 1 and 2% strain of the specimens over a period of about 4 days. On the 14th day after the specimens had been prepared, the repeated loading was stopped and the two specimens were removed from the triaxial compression cells. They were then placed in another cell with provisions for drainage in the cap and base, the confining pressure of 1 kg. per sq. cm. was re-applied and the specimens were loaded to failure in a normal type of strength test. At the same time the two specimens trimmed from the same sample but not subjected to repeated loading were loaded to failure using the same test procedure. The results of three tests of this kind, which are typical of those obtained in a number of other tests, are shown in Figs. 2, 3, and 4. Each curve shown in these figures is the average for two tests performed on similar specimens.

Fig. 2a shows test data for specimens having a water content of 13.5%, a dry density of 105.7 lb. per cu. ft. and a degree of saturation of 60%. In normal triaxial compression tests of the unconsolidated-undrained type but with the ends of the specimens open to the atmosphere, using a lateral pressure of 1 kg. per sq. cm., these specimens had a strength of 6.2 kg. per sq. cm. Specimens subjected to the same confining pressure showed an axial strain of 0.62% when subjected to a deviator stress of 3.65 kg. per sq. cm. for a period of 0.2 seconds and continued to deform when this same stress was repeatedly applied and removed; during the 90,000th application of stress the specimens showed an axial strain of 1.95% and after the stress was removed a residual strain of 1.4%. On being loaded to failure under the same conditions as the previously unloaded specimens these specimens had a strength of 7.75 kg. per sq. cm.

In addition to having a higher strength, the specimens subjected to repeated loading had a different form of stress vs. strain curve from the specimens not subjected to repeated loading, as will be seen from Fig. 2b, where the stress vs. strain characteristics of the specimens during the normal strength tests are plotted from the same origin. The specimens previously subjected to repeated loading were initially stiffer, developed their maximum strength at a lower strain, when deformed beyond the point of maximum strength exhibited a reduced resistance to deformation, and finally had a residual strength approximately equal to that of the previously unloaded specimens.

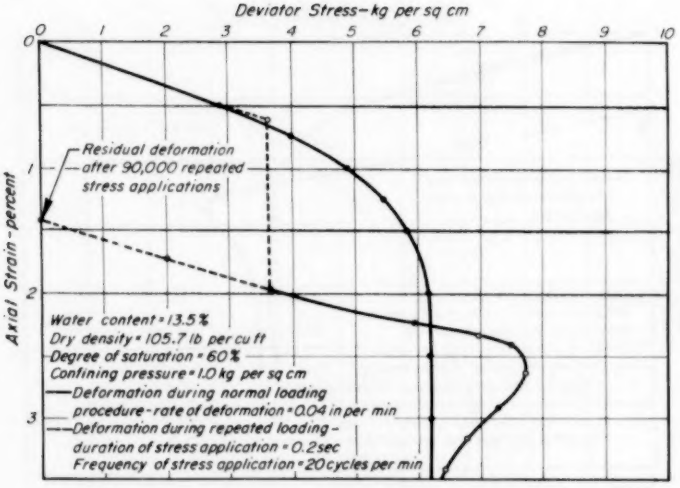

Deviator Stress–kg per sq cm
0 1 2 3 4 5 6 7 8 9 10
Axial Strain - percent
0 1 2 3
Residual deformation after 90,000 repeated stress applications
Water content = 13.5%
Dry density = 105.7 lb per cu ft
Degree of saturation = 60%
Confining pressure = 1.0 kg per sq cm
Deformation during normal loading procedure - rate of deformation = 0.04 in per min
Deformation during repeated loading - duration of stress application = 0.2 sec
Frequency of stress application = 20 cycles per min

Fig. 2(a) - DEFORMATION OF SPECIMENS OF SILTY CLAY IN NORMAL STRENGTH TESTS BEFORE AND AFTER REPEATED LOADING.

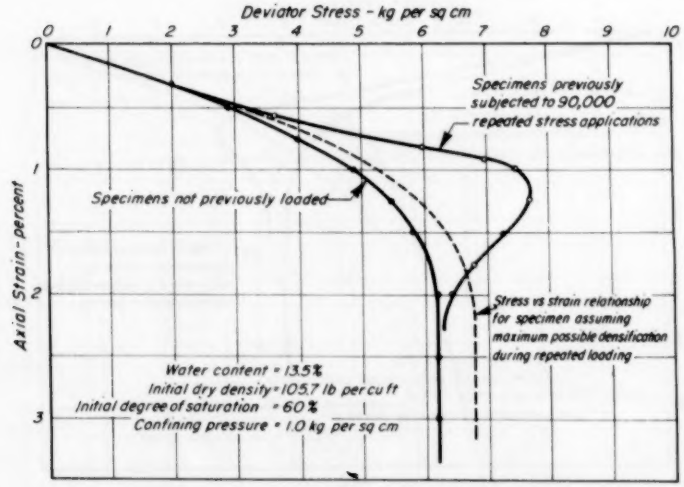

Deviator Stress - kg per sq cm
0 1 2 3 4 5 6 7 8 9 10
Axial Strain - percent
0 1 2 3
Specimens previously subjected to 90,000 repeated stress applications
Specimens not previously loaded
Stress vs strain relationship for specimen assuming maximum possible densification during repeated loading
Water content = 13.5%
Initial dry density = 105.7 lb per cu ft
Initial degree of saturation = 60%
Confining pressure = 1.0 kg per sq cm

Fig. 2(b) - COMPARISON OF STRESS vs STRAIN CURVES OF SPECIMENS IN NORMAL STRENGTH TESTS.

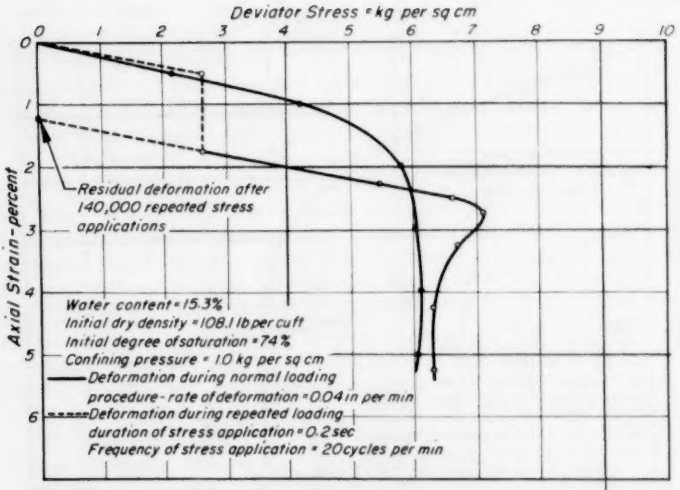

Deviator Stress = kg per sq cm
0 1 2 3 4 5 6 7 8 9 10
Axial Strain - percent
0 1 2 3 4 5 6
Residual deformation after 140,000 repeated stress applications
Water content = 15.3%
Initial dry density = 108.1 lb per cu ft
Initial degree of saturation = 74%
Confining pressure = 1.0 kg per sq cm
Deformation during normal loading procedure - rate of deformation = 0.04 in per min
Deformation during repeated loading duration of stress application = 0.2 sec
Frequency of stress application = 20 cycles per min

Fig. 3(a) - DEFORMATION OF SPECIMENS OF SILTY CLAY IN NORMAL STRENGTH TESTS BEFORE AND AFTER REPEATED LOADING.

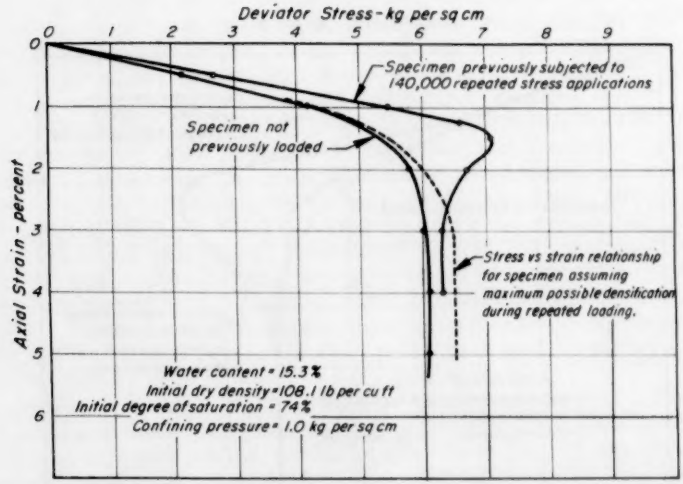

Deviator Stress - kg per sq cm
0 1 2 3 4 5 6 7 8 9
Axial Strain - percent
0 1 2 3 4 5 6
Specimen previously subjected to 140,000 repeated stress applications
Specimen not previously loaded
Stress vs strain relationship for specimen assuming maximum possible densification during repeated loading.
Water content = 15.3%
Initial dry density = 108.1 lb per cu ft
Initial degree of saturation = 74%
Confining pressure = 1.0 kg per sq cm

Fig. 3(b) - COMPARISON OF STRESS vs STRAIN CURVES OF SPECIMENS IN NORMAL STRENGTH TESTS.

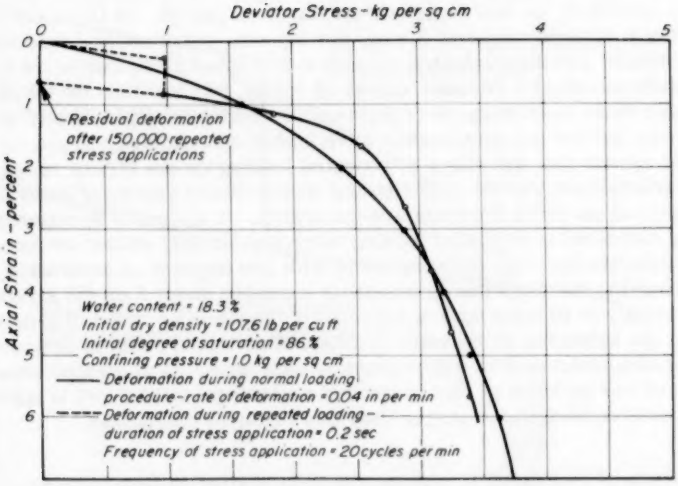

Deviator Stress - kg per sq cm
0
1
2
3
4
5
Axial Strain - percent
Residual deformation after 150,000 repeated stress applications
Water content = 18.3 %
Initial dry density = 107.6 lb per cuft
Initial degree of saturation = 86 %
Confining pressure = 1.0 kg per sq cm
Deformation during normal loading procedure - rate of deformation = 0.04 in per min
Deformation during repeated loading - duration of stress application = 0.2 sec
Frequency of stress application = 20 cycles per min

Fig. 4 (a) - DEFORMATION OF SPECIMENS OF SILTY CLAY IN NORMAL STRENGTH TESTS BEFORE AND AFTER REPEATED LOADING.

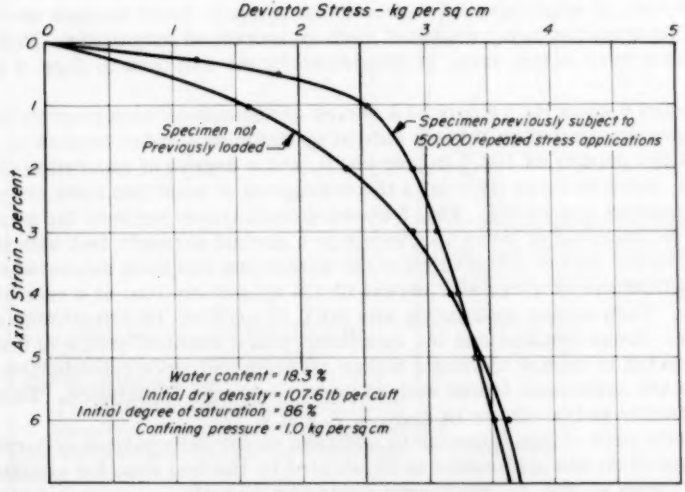

Deviator Stress - kg per sq cm
0
1
2
3
4
5
Axial Strain - percent
Specimen not Previously loaded
Specimen previously subject to 150,000 repeated stress applications
Water content = 18.3 %
Initial dry density = 107.6 lb per cuft
Initial degree of saturation = 86 %
Confining pressure = 1.0 kg per sq cm

Fig. 4 (b) - COMPARISON OF STRESS vs STRAIN CURVES OF SPECIMENS IN NORMAL STRENGTH TESTS.

Similar effects of repeated loading are shown in Figs. 3a and 3b for specimens having a water content of 15.3%, a dry density of 108.1 lb. per cu. ft. and a degree of saturation of 74%.

For specimens having a higher degree of saturation, a somewhat different effect was indicated, as will be seen from Figs. 4a and 4b. In this case specimens having a strength of about 3.5 kg. per sq. cm. under normal loading conditions exhibited a residual deformation of 0.65% after being subjected to 150,000 applications of a deviator stress of 1.0 kg. per sq. cm. On being loaded to failure these specimens were appreciably stiffer than previously unloaded specimens but did not consistently have higher strengths.

It would appear that the effect of repeated loading on the stress vs. deformation characteristics of the soil depends on the initial degree of saturation of the specimen on which the tests are conducted. In all cases specimens previously subjected to repeated loading were appreciably stiffer or more resistant to deformation and, for specimens with low degrees of saturation, had higher strengths, but when the deformation exceeded about 3 or 4% strain exhibited resistances to deformation essentially the same as those of similar specimens not subjected to repeated loading. This would seem to indicate that the increase in resistance to deformation is only apparent at strains less than about 4% and any benefits of this increased resistance will be lost if applied stresses cause deformations larger than this order of magnitude.

Influence of Increased Resistance to Deformation in Determining Soil Behavior Under Repeated Loading

Since the repeated application of stress to a soil specimen causes an increase in stiffness or resistance to deformation of the soil in a normal type of strength test, it might also be expected that the soil would become more resistant to deformation under repeated loads of increased magnitude. That such an effect does in fact occur is illustrated by the test data in Figs. 6 and 7.

Figs. 5 and 6 show the results of a series of unconfined compression tests on specimens compacted on the dry side of optimum at a water content of 16.4% to a dry density of 108.7 lb. per cu. ft. and a degree of saturation of 79%. Each curve in these figures is the average of at least two tests on apparently identical specimens. Fig. 5 shows a comparison between the stress vs. strain relationship of these specimens in a normal strength test with the stress vs. strain curves obtained after the specimens had been subjected to 100,000 applications of a deviator stress of 1.5 kg. per sq. cm. at a rate of 20 per minute. Each stress application was for 0.25 second. On completion of the repeated stress applications the specimens had a residual strain of 0.95% and when loaded to failure showed a higher strength and were considerably stiffer than the specimens tested with no previous stress application. This result is similar to that shown in Figs. 2, 3, and 4.

The significance of this increase in stiffness on the deformation occurring during repeated stress application is illustrated by the test data for similar specimens shown in Fig. 6. Specimens subjected directly to a repeated stress of 2.9 kg. per sq. cm. at a rate of 20 applications per minute (each application having a duration of 0.25 second) deformed slightly more than 1% during the first application, continued to deform with each subsequent application, and failed completely after about 100 stress applications. However, the specimens

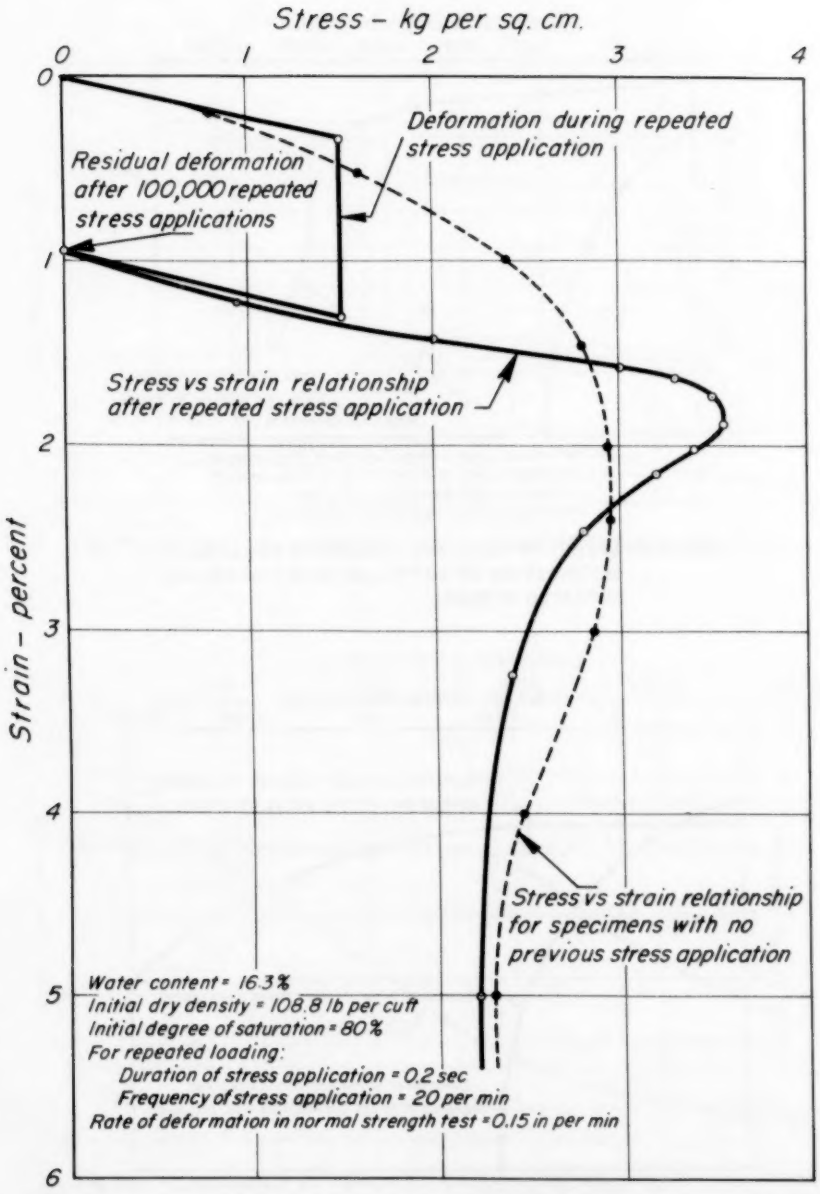

Stress – kg per sq. cm.
0
1
2
3
4
Strain – percent
0
1
2
3
4
5
6
Deformation during repeated stress application
Residual deformation after 100,000 repeated stress applications
Stress vs strain relationship after repeated stress application
Stress vs strain relationship for specimens with no previous stress application
Water content = 16.3%
Initial dry density = 108.8 lb per cuft
Initial degree of saturation = 80%
For repeated loading:
Duration of stress application = 0.2 sec
Frequency of stress application = 20 per min
Rate of deformation in normal strength test = 0.15 in per min

Fig.5- EFFECT OF REPEATED LOADING ON STRESS vs DEFORMATION RELATIONSHIP OF SILTY CLAY IN UNCONFINED COMPRESSION TESTS.

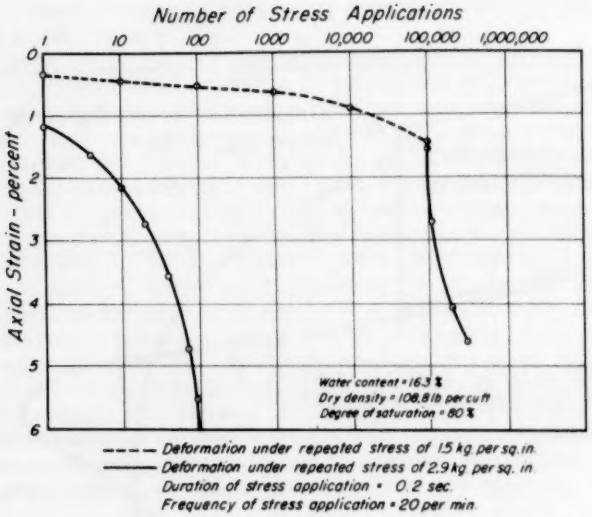


Fig. 6 (a) - EFFECT OF REPEATED LOADING ON SUBSEQUENT DEFORMATION OF SILTY CLAY UNDER INCREASED REPEATED STRESS.

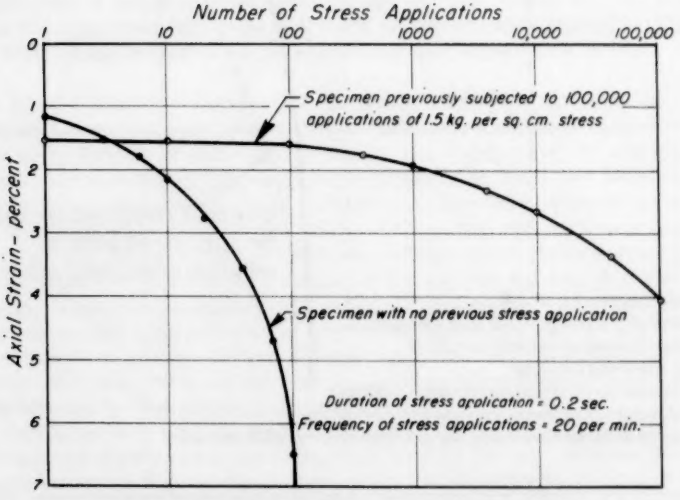


Fig. 6 (b) - COMPARISON OF SPECIMEN DEFORMATIONS UNDER REPEATED STRESS OF 2.9 KG PER SQ CM.

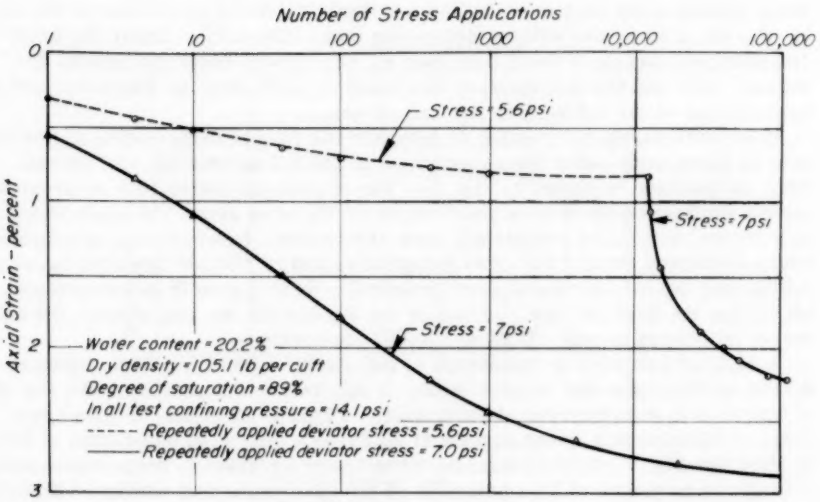


Fig. 7(a) - EFFECT OF CHANGE IN LOAD DURING REPEATED LOADING.

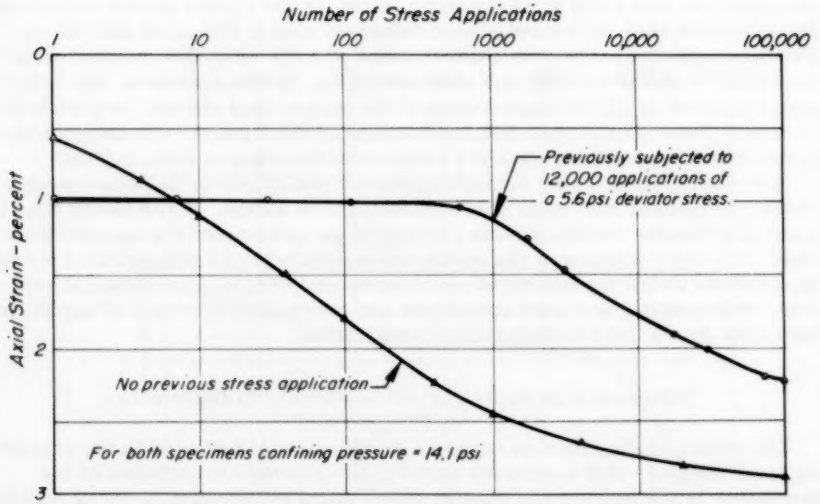


Fig. 7(b) - COMPARISON OF SPECIMEN DEFORMATIONS UNDER REPEATED DEVIATOR STRESS OF 7 psi .

previously subjected to 100,000 applications of a 1.5 kg. per sq. cm. stress were subsequently able to withstand a further 200,000 applications of the 2.9 kg. per sq. cm. stress without deforming more than 4.6%. Under the first 100,000 applications of the 1.5 kg. per sq. cm. stress these specimens deformed 1.4% and the deformation increased to 4.6% after an additional 200,000 applications of the 2.9 kg. per sq. cm. stress.

It is particularly interesting to compare the rate of deformation of the two sets of specimens under the applications of the 2.9 kg. per sq. cm. stress. This comparison is shown in Fig. 6b. The specimens tested with no previous stress application showed an axial strain of 7% after about 100 applications of this stress, and failed completely soon afterwards. However, the specimens which deformed about 1.5% after being subjected to 100,000 applications of the 1.5 kg. per sq. cm. stress showed practically no increase in deformation at all during the first 100 applications of the 2.9 kg. per sq. cm. stress; the axial strain increased to only 4% after 100,000 applications.

A similar increase in resistance to deformation resulting from repeated stress applications was found to occur in specimens compacted on the wet side of optimum to high degrees of saturation. A typical example, for two specimens compacted at a water content of 20.2% to a degree of saturation of 89% is shown in Fig. 7. Both specimens were tested in triaxial compression using a confining pressure of 14.1 psi. One of the specimens was subjected directly to repeated applications of a 7 psi deviator stress, each stress application having a duration of 0.25 second, at a rate of 10 applications per minute. The second specimen was subjected first to 12,000 applications of a 5.6 psi deviator stress of the same duration and frequency and then the repeated deviator stress was increased to 7 psi as for the first specimen. It will be seen that the specimen subjected to 12,000 applications of the lighter stress before being subjected to an increased stress deformed only 2.25% after 100,000 applications, as compared with a deformation of 2.9% after this number of applications to the previously unloaded specimen. In this case also, the specimen subjected to 12,000 applications of the lighter load showed very little increase in deformation after 500 applications of the 7 psi stress, though subsequent stress applications caused a substantial increase in deformation.

It is worthy of note that no such increased resistance to deformation as a result of repeated loading is observed in tests on sands. Fig. 8 shows the results of a similar series of tests performed on specimens of a medium fine sand. The deformations of the specimens subjected to 22,000 repeated stress applications under the increased deviator stress were almost identical with those of previously unloaded specimens and there is no evidence of any stiffening effect due to the previous stress applications.

Influence of Densification on Resistance to Deformation

The nature of the effect of repeated loading on soil behavior in the preceding tests suggests that a primary cause of the increase in stiffness of the specimens is the increase in density which would be expected to occur during the repeated application of loads. In order to determine the significance of an increase in density on the deformation characteristics of the soil, a series of tests were conducted on specimens at various water contents and densities in order that the behavior of a specimen at any water content and density might be determined by interpolation.

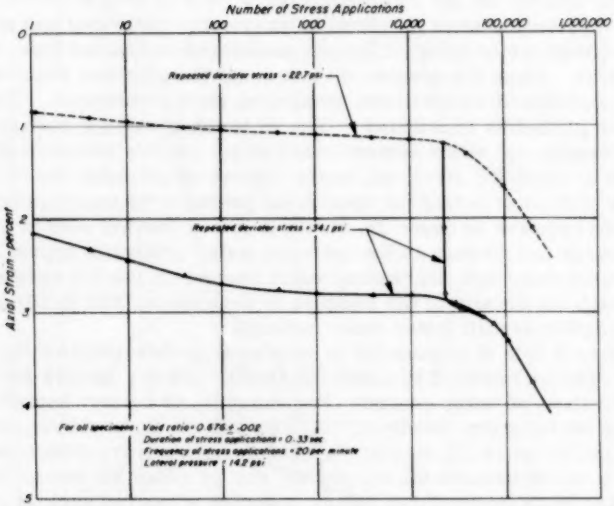


Fig. 8a – EFFECT OF INCREASE IN STRESS ON FINE SAND DURING REPEATED LOADING.

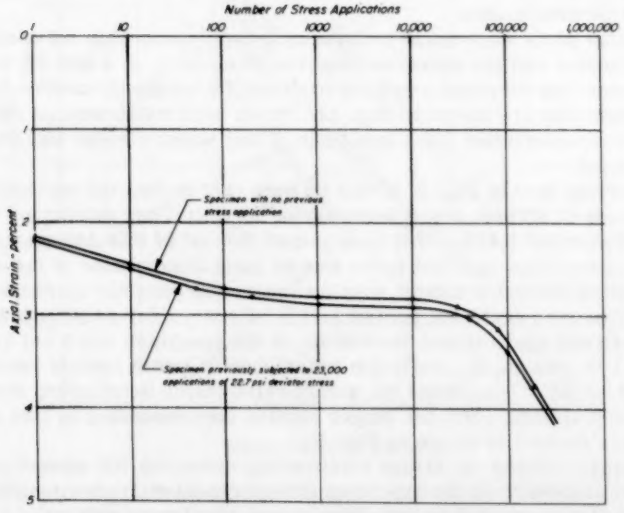


Fig. 8b – COMPARISON OF DEFORMATIONS OF FINE SAND SPECIMENS UNDER REPEATED DEVIATOR STRESS OF 34.1 psi

For this purpose series of tests were first made to establish the relationship between density and water content and between resistance to deformation and water content for the soil compacted at each of four different compactive efforts. Specimens were prepared as previously described and subjected to triaxial compression tests of the unconsolidated-undrained type, 14 days after preparation. From the results of these tests the stresses required to cause various amounts of strain of the specimens were determined. The results were then plotted as illustrated in Fig. 9, which shows for each compactive effort the density vs. water content relationship and the corresponding stress required to cause 2% strain vs. water content relationship for the same specimens. It will be seen that for specimens prepared by each compactive effort the stress required to cause 2% strain is low on the wet side of optimum, but that it begins to increase as the optimum water content is approached, continues to increase with decreasing water content on the dry side of optimum even though the density of the samples is decreasing, and finally begins to decrease slightly at still lower water contents.

From such data it is possible to determine by interpolation the relationship between "stress required to cause 2% strain" and dry density for various constant values of water content. For example, at a water content of 14% a specimen having a dry density of 100.5 lb. per cu. ft. would require 3.8 kg. per sq. cm. to cause 2% strain, a specimen having a dry density of 106.1 lb. per cu. ft. would require 6.2 kg. per sq. cm. to cause 2% strain, and so on. By plotting these interpolated values, a series of curves such as those shown in Fig. 10 is obtained. It will be seen that the stress required to cause 2% strain does not necessarily increase as the density of specimens increases but may increase or decrease depending on both the water content and the range of densities concerned. From such a series of curves the stress required to cause 2% strain for a specimen of any water content and density can readily be determined.

Similar plots were made to determine the relationship between density, water content and the stresses required to cause 1, 3, 4 and 5% strain of the specimens; the stresses required to cause 1% strain at various densities and water contents are shown in Fig. 11. From such relationships the stress vs. strain characteristics for a specimen of any water content and density can be determined.

Referring now to Fig. 2, it will be seen that during the application of 90,000 repetitions of stress, a specimen having an initial dry density of 105.7 lb. per cu. ft. deformed 1.41%. If it is assumed that all of this deformation is the result of compaction and that there was no lateral expansion of the specimen accompanying the deformation, then the maximum possible increase in density of the specimen could not exceed 1.41%. Hence, after being subjected to 90,000 stress applications the density of the specimen could not have exceeded 107.2 lb. per cu. ft., while the water content would remain unchanged at a value of 13.5%. The stress vs. strain relationship determined from Figs. 10 and 11 and similar plots for larger strains corresponding to this new density and water content is shown in Fig. 2b.

A similar stress vs. strain relationship assuming the maximum possible increase in density of the specimen during repeated loading is shown in Fig. 3b. It will be seen that in both these cases specimens subjected to repeated loading have greater stiffness and higher strengths than could possibly occur in specimens having the same densities and water contents but not previously subjected to repeated loading.

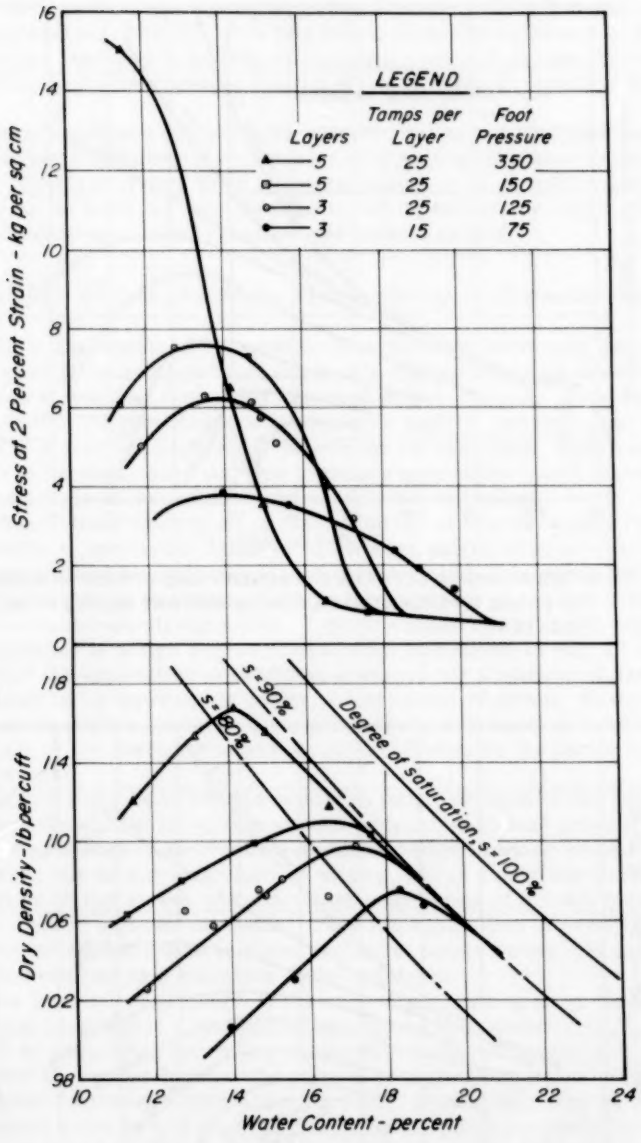


Fig.9- RELATIONSHIP BETWEEN DRY DENSITY, WATER CONTENT, AND STRESS REQUIRED TO CAUSE 2 PERCENT STRAIN FOR COMPACTED SILTY CLAY.

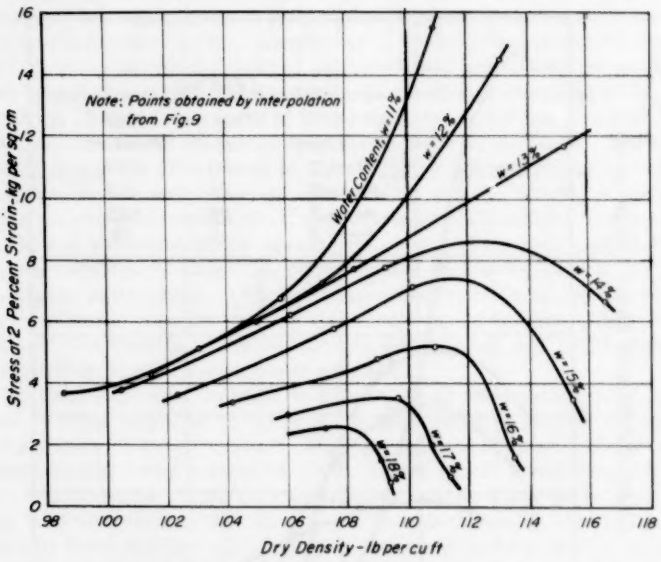


Fig.10-RELATIONSHIP BETWEEN DRY DENSITY AND STRESS REQUIRED TO CAUSE 2 PERCENT STRAIN FOR CONSTANT VALUES OF WATER CONTENT.

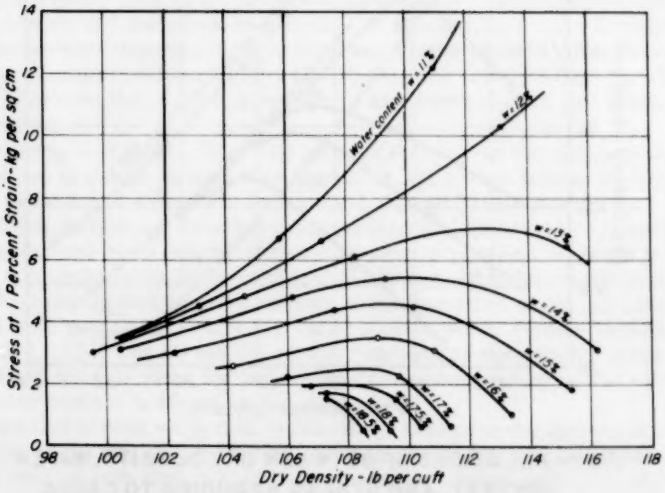


Fig.11 - RELATIONSHIP BETWEEN DRY DENSITY AND STRESS REQUIRED TO CAUSE 1 PERCENT STRAIN FOR CONSTANT VALUES OF WATER CONTENT.

No stress vs. strain curve to show the effect of densification is presented in Fig. 4b for the specimen having an initial density of 107.6 lbs. per cu. ft. and a water content of 18.3%. For this initial condition reference to Fig. 11 shows that any increase in density will cause a reduction in resistance to deformation. Yet after repeated loading the specimens were stiffer than before.

It would appear from these results that the increase in stiffness and sometimes in strength resulting from a series of stress applications to the soil cannot be attributed simply to an increase in density of the soil during repeated loading; or at least it cannot be determined by normal strength tests on compacted samples of equal densities and water contents.

Influence of Sustained Pressure on Deformation Characteristics

During the application of repeated load in the tests previously described, the specimens were subjected to a constantly applied confining pressure, while the deviator stress was repeatedly removed and re-applied. In cases where as many as 100,000 repetitions of stress were applied, the specimen was thus subjected to a constant confining pressure for several days. Consideration was given to the possibility that this pressure application might be responsible for the change in deformation characteristics of the soil.

Accordingly tests were made to determine the stress vs. strain relationship of identical specimens, some of which were subjected to a constant confining pressure and immediately loaded to failure, while others were maintained under constant confining pressure for a number of days and then loaded to failure in a normal strength test. Typical results of such tests on specimens compacted to a high degree of saturation are shown in Fig. 12. It will be seen that the application of confining pressure for a number of days causes the specimen to be appreciably stiffer at low values of strain. However, the influence of sustained pressure disappears as the deformation increases, and the strength of the specimen is practically unaffected by the period of sustained pressure.

This effect might be attributed in part to minor changes in the distribution of moisture throughout the specimens subjected to sustained pressure. In a compacted soil there is inevitably a small variation in water content throughout the mass due to non-uniformity of mixing, and in a partially saturated soil there will be higher stress concentrations near points of contact between larger grains than in adjacent air voids. Thus the application of pressure is likely to cause small decreases in water content at points throughout the soil structure, resulting in a stiffening of the specimen.

Another possible explanation of the stiffening resulting from the application of confining pressure is a mechanism similar to that advanced by T. W. Lambe[3] in describing secondary compression effects in saturated clays: "Even after the excess hydrostatic pressures in the pore water are dissipated the particles continue to approach each other. During this very slow process the adsorbed water is extruded from between the particles and to some extent becomes pore water. While this process can occur with negligible volume change. . . .the extrusion or diffusion of adsorbed water may require many years to be completed. The transfer of adsorbed to pore water results in a strength increase since the particles are closer together at points of contact". Such a gradual moving together of the particles may well occur in compacted clay resulting in a stiffening of the soil.

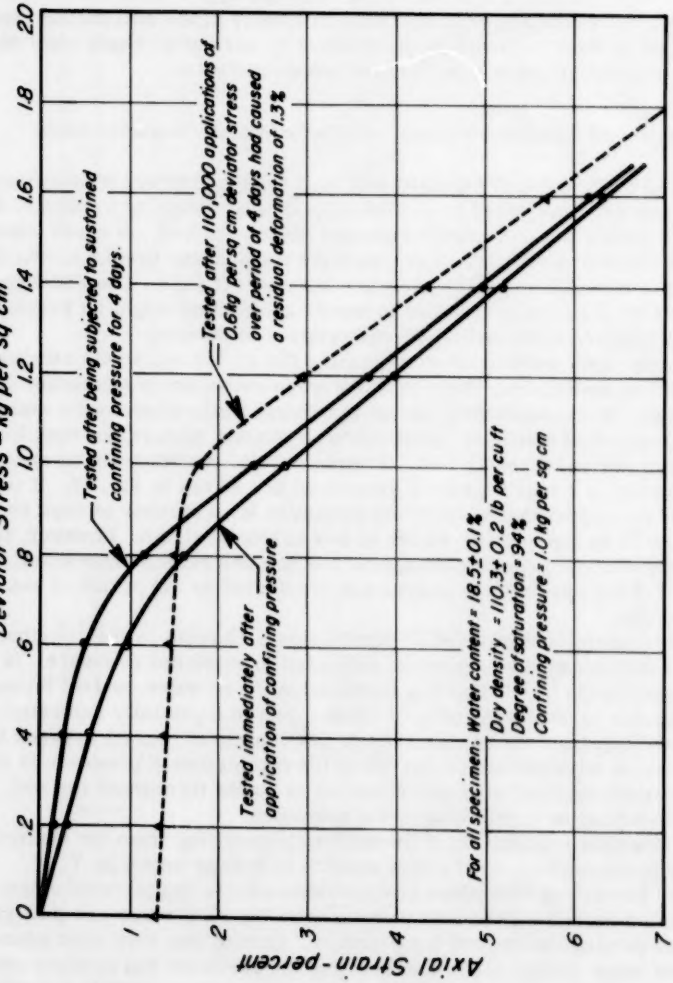


Fig.12 – EFFECT OF SUSTAINED CONFINING PRESSURE ON STRESS vs STRAIN RELATIONSHIP FOR SILTY CLAY IN NORMAL STRENGTH TESTS.

As in the case of moisture redistribution within a specimen, the major diffusion of adsorbed water is likely to occur at points of stress concentration. Thus if the particles at these points are re-separated by large deformations this source of increased resistance to deformation is likely to be lost. Such a loss was in fact observed in the tests shown in Figs. 2, 3, and 4.

While the stiffening of the specimens due to application of a sustained confining pressure is undoubtedly partially responsible for the observed increase in stiffness of the specimens in the triaxial compression tests under repeated load, it cannot alone be responsible for the increased stiffness observed in these tests, since, as will be seen in Fig. 12, specimens subjected to repeated stress applications are considerably stiffer and stronger than those subjected to sustained confining pressure for an equal period of time. Furthermore, as shown in Fig. 5, there is a considerable increase in stiffness and strength of specimens subjected to repeated axial stress even when no confining pressure is applied.

Possible Causes of Increased Resistance to Deformation

Since the observed increase in stiffness of soil specimens during repeated loading tests cannot be entirely attributed to the influence of densification or of the confining pressure, other possible changes in the characteristics of the specimens must be considered.

Examination of the test data would seem to indicate that specimens subjected to greater deformations during repeated loading subsequently possess the greater resistance to deformation. However, the larger deformations are necessarily associated with greater magnitudes and numbers of applications of the repeated stress and it is believed that these two factors rather than the resulting strain are primarily responsible for the increased stiffness of the specimen.

The significance of the number of stress applications is illustrated by the test data presented in Fig. 13. In this series of tests three specimens trimmed from the same compacted sample were subjected to repeated loads of different magnitudes until each of the specimens had an axial strain of 3.55%. When this strain was reached the specimens were loaded to failure as in a normal strength test.

For the specimens subjected to the highest repeated stress, 8,000 applications were required to produce this deformation; for the specimens subjected to lighter loads, 30,000 and 350,000 applications were required respectively. The progress of deformation during repeated loading is shown in Fig. 13a. The stress vs. strain curves of the specimens during the normal loading procedure are shown in Fig. 13b. It will be seen that although the three specimens had equal initial deformations after repeated loading, the specimens subjected to the greater number of stress applications had the highest stiffness and strength, while the specimens subjected to the least number of stress applications had the least resistance to further deformation.

Thus a large number of lighter stress applications appear to have a greater effect on the subsequent behavior of a soil than a smaller number of applications of a higher stress. However, reference to Fig. 1 shows that for specimens having the same number of repeated stress applications the larger the magnitude of the repeated stress the greater is the increase in strength. Thus the observed increase in stiffness would appear to be associated primarily

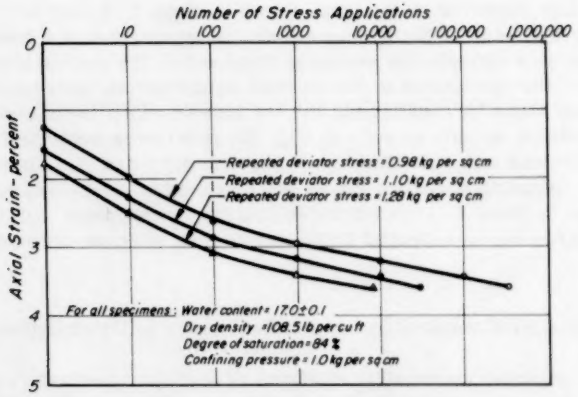


Fig. 13(a) DEFORMATION OF SPECIMENS OF SILTY CLAY UNDER REPEATED LOADING.

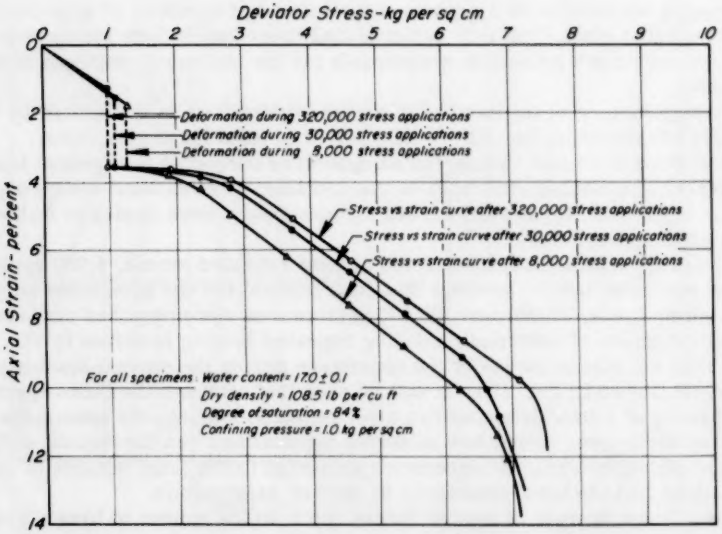

Deviator Stress - kg per sq cm
0
1
2
3
4
5
6
7
8
9
10
Axial Strain - percent
Deformation during 320,000 stress applications
Deformation during 30,000 stress applications
Deformation during 8,000 stress applications
Stress vs strain curve after 320,000 stress applications
Stress vs strain curve after 30,000 stress applications
Stress vs strain curve after 8,000 stress applications
For all specimens: Water content = 17.0 ± 0.1
Dry density = 108.5 lb per cu ft
Degree of saturation = 84 %
Confining pressure = 1.0 kg per sq cm

Fig. 13(b)-EFFECT OF NUMBER AND MAGNITUDE OF STRESS APPLICATIONS ON SOIL DEFORMATION CHARACTERISTIC.

with the number and magnitude of the repeated stress applications.

Since the effect is not due to densification, it would appear that the change in deformation characteristics must be attributed to other changes occurring within the specimen as a result of repeated loading and is possibly associated with changes in moisture distribution or structural arrangement of the soil grains or with an increase in resistance to deformation caused by thixotropy.

Changes in moisture distribution might be expected to occur in a similar fashion to that previously described in discussing the effects of sustained confining pressures. The application of a deviator stress might be expected to cause stress concentrations near points of contact between larger grains and, when the deviator stress application is repeated a large number of times, the moisture in these zones of stress concentration will flow to adjacent areas of lower stress within the specimen, causing a stiffening of the clay adjacent to the larger grains. This in turn will result in the development of a soil structure consisting of larger particles and stiffer clay with a somewhat weaker clay forming the matrix between the main skeleton. The increased resistance to deformation would then be attributed to the development of this type of soil structure and would be lost if the structure were disturbed by excessive deformation. Such a loss at high strains was in fact observed in the tests (see page 6).

Another effect which may be associated with large numbers of stress applications is the development of thixotropic strength during the period of stress applications. It has previously been indicated[2] that a thixotropic material may lose strength as a result of the deformation caused by application of a deviator stress and subsequently regain part of this strength if it is allowed to rest without any further appreciable deformation. This fact is particularly important in clay samples having high degrees of saturation. In the tests previously described the soil deformed very little after the first several thousand applications and thus might possibly have increased in stiffness as a result of thixotropic strength regain during the subsequent period of stress applications in which very little deformation was taking place. As in the case of the deformation of the soil structure discussed above, any increase in stiffness due to this cause would tend to disappear if the specimen were deformed sufficiently by subsequent loads.

Finally there exists the possibility that a series of repeated stress applications might cause changes in the structural arrangement of grains in the compacted specimens. The increased resistance to deformation appears to be developed by a large number of applications of stress over a period during which the soil deforms very little. Thus, as previously described, the repeated pressure application may well cause particles to move slightly closer together by extrusion of adsorbed water with a resulting increase in strength. For a further discussion of the influence of pressure and time on the structure and strength of compacted clays the reader is referred to a recent paper by T. W. Lambe.[4]

It is important to recognize that the increased resistance to deformation appears to be lost if the soil is deformed appreciably after the increased resistance has been developed. This is apparent in Figs. 2 and 3, which show that the residual strength of the specimens previously subjected to repeated loading is about the same as that of previously unloaded specimens and is reached at axial strains of 2 or 3 per cent. Again in Fig. 4, the increased resistance to deformation of specimens previously subjected to repeated loading is apparently lost when they subsequently reach a deformation of about 4 per cent.

The loss of the increased resistance developed during repeated loading when the soil is subsequently deformed appreciably is particularly well illustrated by the data in Figs. 14 and 15. In these tests specimens were first subjected to 30,000 applications of an axial stress of 0.4 kg. per sq. cm. On some specimens the axial stress was then increased to 0.5 kg. per sq. cm., on others to 0.7 kg. per sq. cm. and on still others to 1.0 kg. per sq. cm.

The deformations of these specimens under further repeated stress applications were then compared with those of specimens subjected to the higher stresses without previous load application. That the 30,000 applications of the 0.4 kg. per sq. cm. axial stress caused some stiffening of the soil is apparent from a comparison of the deformations occurring under repeated applications of the 0.5 kg. per sq. cm. axial stress. Even after 30,000 applications of this stress the previously loaded specimens had deformed considerably less than previously unloaded specimens.

However for those specimens on which the repeated axial stress was changed to 0.7 kg. per sq. cm. the effect of the previous stiffening is less apparent and, as will be seen from Figs. 14 and 15, when the repeated stress was increased to 1.0 kg. per sq. cm. the stiffening effects of the previous stress application disappeared completely. In addition to indicating the nature of the effect, these data also illustrate the necessity of avoiding large stress changes if the increased resistance is to be used to practical advantage.

In general then it appears that a large number of stress applications can cause an increased resistance to deformation in a soil having a constant water content provided that the soil does not deform excessively during these applications. This increased stiffness is due partly to minor changes in moisture distribution in the soil and to thixotropic effects. However, it is probably mainly due to a decreased spacing between clay size particles resulting from the application of repeated stresses or a sustained confining pressure. The increased stiffness is destroyed by large deformations probably because such deformations cause a re-separation of particles or because the particle spacing is only changed at points of stress concentration and is obscured when large deformations cause the soil at these points to be mixed with particles in relatively unaffected zones.

CONCLUSION

In practice, elements of soil subjected to repeated axial stress application will simultaneously be subjected to repeated lateral stresses. This condition can be considered as the repeated application of a confining pressure and the simultaneous application of a deviator stress. In the tests described above the confining pressure was maintained constant, and to this extent the loading conditions are different from those normally encountered. However, since the increased resistance to deformation is apparently primarily due to the repeated application of the deviator stress, it is believed that the results obtained will be essentially similar to those which will develop under more realistic loading conditions.

The test data show that a large number of applications of stress to a clay soil can cause an increase in resistance to deformation. The number of applications required to produce this effect was not determined but there were some indications that it is certainly more than 1,000. Furthermore, in all of the tests conducted the deformation during the repeated stress applications

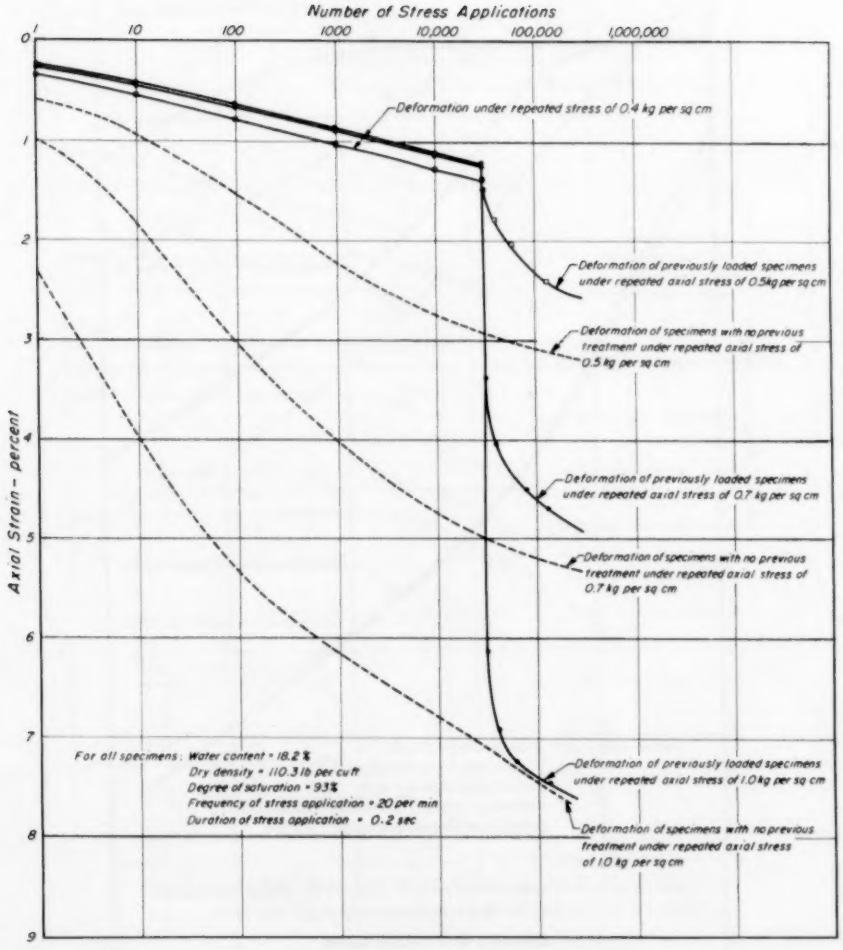

Number of Stress Applications
1
10
100
1000
10,000
100,000
1,000,000
0
1
2
3
4
5
6
7
8
9
Axial Strain - percent
Deformation under repeated stress of 0.4 kg per sq cm
Deformation of previously loaded specimens under repeated axial stress of 0.5kg per sq cm
Deformation of specimens with no previous treatment under repeated axial stress of 0.5 kg per sq cm
Deformation of previously loaded specimens under repeated axial stress of 0.7 kg per sq cm
Deformation of specimens with no previous treatment under repeated axial stress of 0.7 kg per sq cm
Deformation of previously loaded specimens under repeated axial stress of 1.0 kg per sq cm
Deformation of specimens with no previous treatment under repeated axial stress of 1.0 kg per sq cm
For all specimens: Water content = 18.2 %
Dry density = 110.3 lb per cu ft
Degree of saturation = 93%
Frequency of stress application = 20 per min
Duration of stress application = 0.2 sec

Fig. 14 – EFFECT OF MAGNITUDE OF STRESS CHANGE ON DEFORMATION OF SILTY CLAY UNDER REPEATED LOADING – UNCONFINED COMPRESSION TESTS.

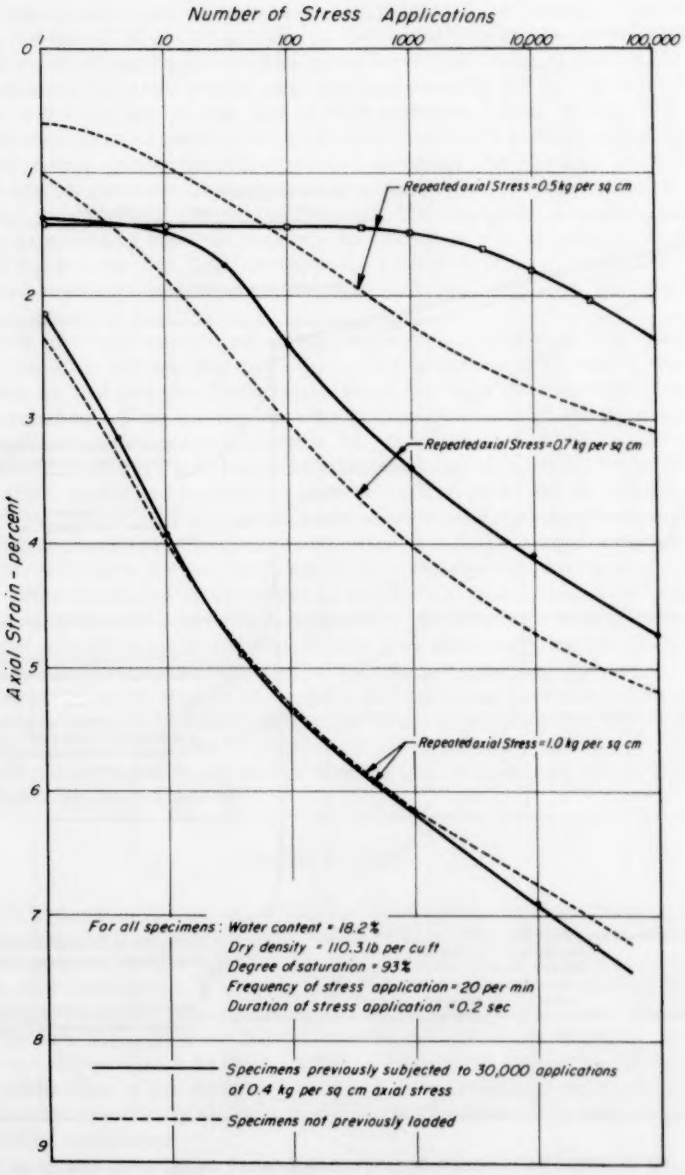


Fig. 15 - EFFECT OF MAGNITUDE OF STRESS CHANGE ON DEFORMATION OF SILTY CLAY UNDER REPEATED LOADING - UNCONFINED COMPRESSION TESTS.

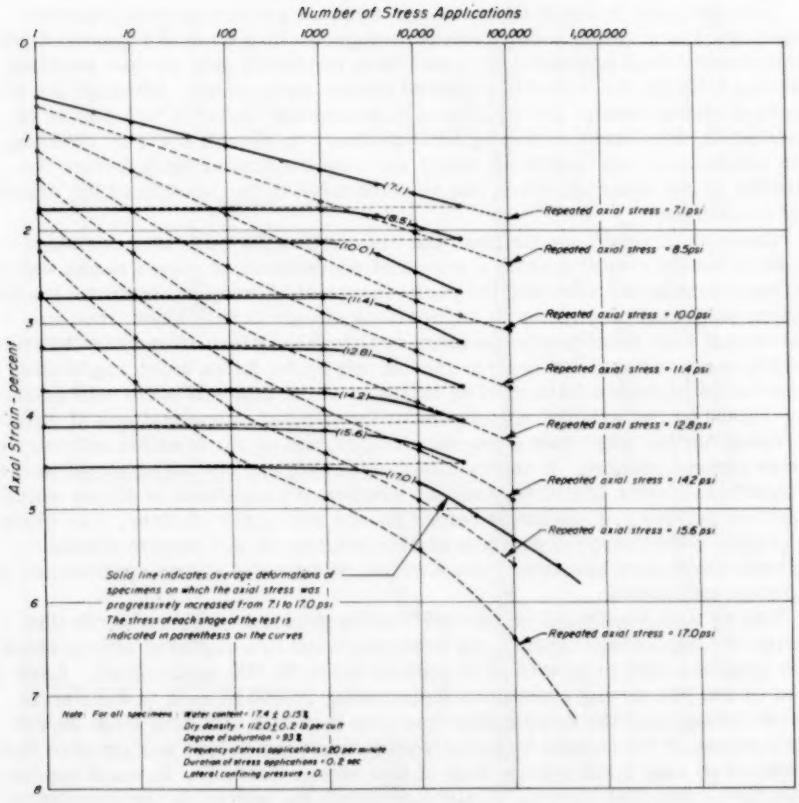


Fig.16- DEFORMATIONS OF SPECIMENS OF SILTY CLAY UNDER PROGRESSIVELY INCREASED REPEATED STRESS.

were relatively small. These facts are worthy of emphasis because under other conditions a series of applications of stress may result in a decreased resistance to deformation. If the magnitude of the repeated stress is sufficiently large to cause an increase in density and the soil is gradually compacted by repeated applications of the stress to a high degree of saturation, a decrease in strength will occur.[5,6] This condition would appear to be associated with somewhat larger deformations of the soil during loading and smaller numbers of stress applications. Whether there is an increase or decrease in resistance to deformation is likely to depend, therefore, on the magnitude and number of the repeated stress applications.

It should also be noted that the tests were all performed on specimens maintained at constant water content throughout the period of repeated loading, whereas in actual practice a soil will often be able to gain or lose water by drainage during the period of repeated stress applications. Although the effects of stress history are evidenced in specimens having a high degree of saturation, and therefore having little tendency to absorb water by swelling, the influence of this additional factor will require further study before the validity of the observed effect can be considered to be established for practical conditions.

However, it would appear from the results that the increased resistance to deformation resulting from a series of applications of stress might well be an important factor affecting the performance of highway pavements. If a clay soil is subjected to a series of applications of light traffic loads, then apparently it may subsequently be more resistant to deformation under heavier traffic loads and may not deform excessively under loads which might otherwise cause pavement failure. The extent to which this will occur will probably depend on the relative magnitudes of the number of applications of different stresses, on the magnitude of stress changes, and on the possible influence of water content.changes. It seems likely, however, that the effect might well be a significant factor in determining the number of repetitions of stress which might be applied to a pavement before failure will occur; further, it is likely to greatly complicate the problem of determining the destructive effects on highway pavements resulting from a series of repeated stress applications of different intensities.

This is well illustrated by the test results presented in Fig. 16. In this series of tests several specimens were subjected to a repeated stress which was progressively increased in magnitude every 30,000 applications. After a total of 240,000 stress repetitions (comprising 30,000 at each of 8 different stress intensities) the deformation was less than that resulting from 30,000 applications of the highest stress intensity alone and in fact was equal to that produced by only 1,500 applications of this stress intensity. It would appear from these data that there is no simple method for assessing the cumulative effects of a series of wheel loads of different intensities on clay soils from data concerning their individual effects.

The practical difficulty of determining the service life of a highway pavement is illustrated by the wide variation in the predictions of the various State Highway Departments of the number of load repetitions which would cause failure in the WASHO test road at Malad, Idaho.[7] After being provided with samples of the soil, each State Highway Department was asked to predict the number of repetitions of a given axel load which would cause failure. The collected results of these predictions are shown in Fig. 17. Clearly there is a wide variation in the experiences of the life of a highway pavement in different parts of the country.

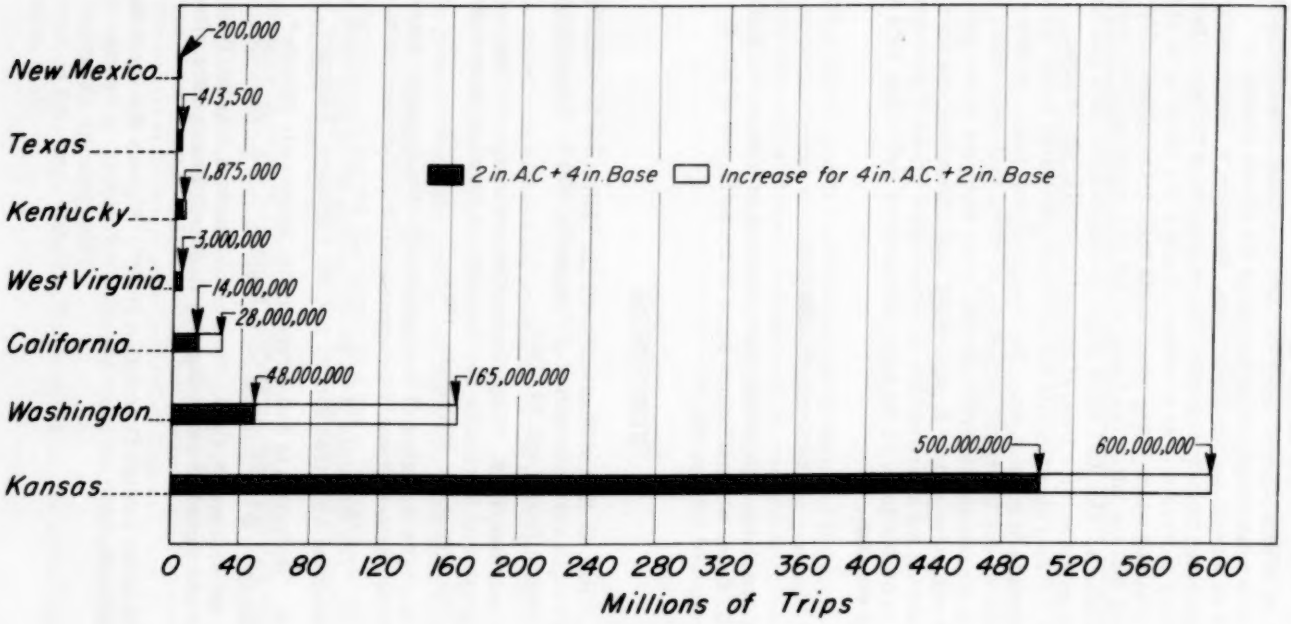


Fig.17 - NUMBER OF TRIPS TO PRODUCE FAILURE, 18,000-lb SINGLE AXLE LOAD, 16-in. SUBBASE.

(from Highway Research Board Bulletin No 133)

There are, of course, many factors to be included in such a prediction, and it is not the intent of this paper to infer that the variations in the predictions shown in Fig. 15 are due simply to variations in the stress history of pavements used as a basis for the predictions. However, it would appear that experiences with older highways on which the magnitude of the wheel loads has been gradually increased over a number of years are likely to be widely different from those on which heavy wheel loads have been imposed soon after construction, and that the latter are likely to fail at a much smaller number of stress applications. This fact alone might be responsible for a substantial difference between the service lives of otherwise similar pavements.

Finally, the test data also indicate the extreme difficulties likely to be encountered in attempts to predict, from the results of soil tests, the deformation of pavements due to erratic sequences of wheel load intensities; the care required in translating experiences with older highways to new pavements which are to be subjected mainly to high wheel load intensities; and the desirability of considering the entire pattern of stress applications if pavements are eventually to be designed, on the basis of a limiting deflection, with maximum safety and economy.

ACKNOWLEDGEMENT

The authors gratefully acknowledge the assistance of Messrs. C. K. Chan, L. Shifley and G. Baker who performed some of the tests, and of Mr. G. Dierking who prepared the figures.

REFERENCES

1. Seed, H. B., Chan, C. K. and Monismith, C. L., "Effects of Repeated Loading on the Strength and Deformation of Compacted Clay". Proceedings, Highway Research Board, vol. 34, 1955.

2. Seed, H. B. and Chan, C. K., "Thixotropic Characteristics of Compacted Clays". Journal, Soil Mechanics and Foundations Division, American Society of Civil Engineers, November, 1957.

3. Lambe, T. W., "The Structure of Inorganic Soil". Proceedings, American Society of Civil Engineers, vol. 79, Separate No. 315, October, 1953.

4. Lambe, T. W., "The Structure of Compacted Clay", paper presented at Chicago Convention American Society of Civil Engineers, February, 1958.

5. Foster, C. R., "Reduction in Soil Strength with Increase in Density", Transactions, American Society of Civil Engineers, vol. 120, 1955.

6. Seed, H. B. and Monismith, C. L., "Some Relationships Between Density and Stability of Subgrade Soils". Highway Research Board, 1954. (Bulletin No. 93.)

7. "Flexible Pavement Design Correlation Study". Highway Research Board, 1956. (Bulletin No. 183.)

Paper 1646 May, 1958 SM 2

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

GEOLOGY AND SOILS OF THE NEWARK (N.J.) METROPOLITAN AREA[a]

Alfreds R. Jumikis,[1] M. ASCE
(Proc. Paper 1646)

SUMMARY

This article outlines a short geological sketch of the area, reports about the major soil types encountered and mapped in the glaciated Newark metropolitan area of New Jersey, indicates some practical applications of the engineering soil maps, and presents some physical properties of the glacial soils.

INTRODUCTION

Purpose of Article

The purpose of this article is

1. to inform the engineering profession on the recently completed engineering soil survey of New Jersey by Rutgers, The State University, for the State of New Jersey;
2. to present briefly the geology and soil types associated with it in the general Newark metropolitan area of New Jersey. This area is of particular engineering significance because through it all of the major interstate highways radiate out of and into New York, carrying the world's largest traffic volume. In this respect, New Jersey may be considered a heavily travelled corridor state;
3. to describe to some extent the methods and procedures used in executing the engineering soil survey and in preparing the engineering soil maps;

Note: Discussion open until October 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1646 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 2, May, 1958.

a. Presented at the New York Convention of the American Society of Civil Engineers, October, 1957.

1. Prof. of Civ. Eng., Rutgers, The State Univ., College of Eng., New Brunswich, N. J.

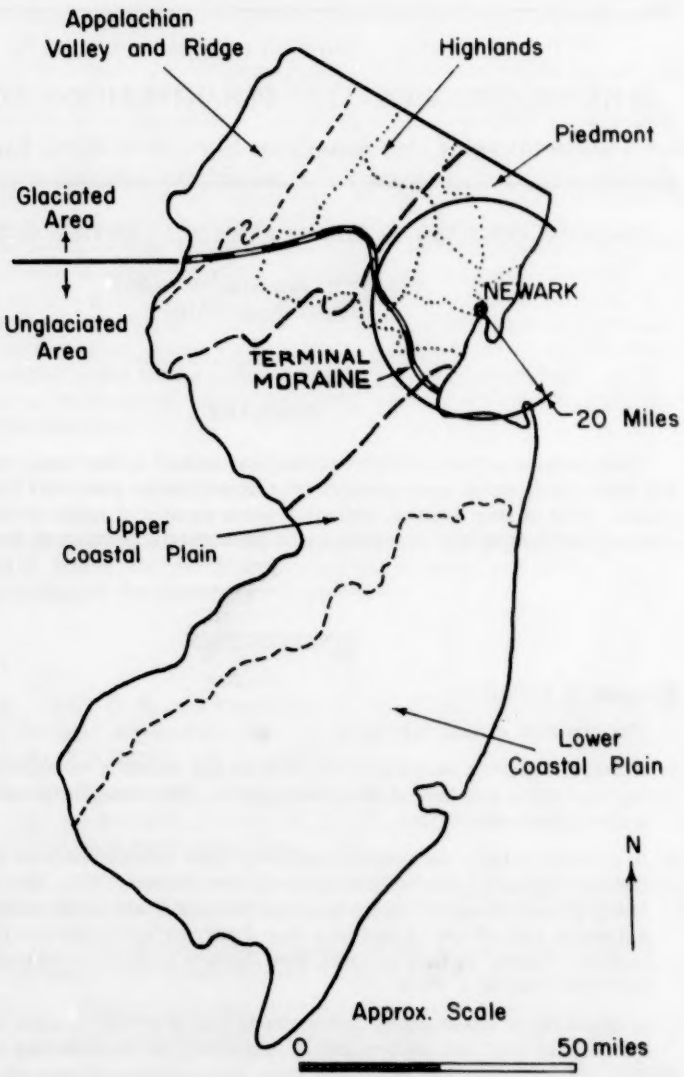


FIG. 1-A Sketch Map Showing the Newark Metropolitan Area of New Jersey, the Principal Physiographic Provinces, and the Terminal Moraine.

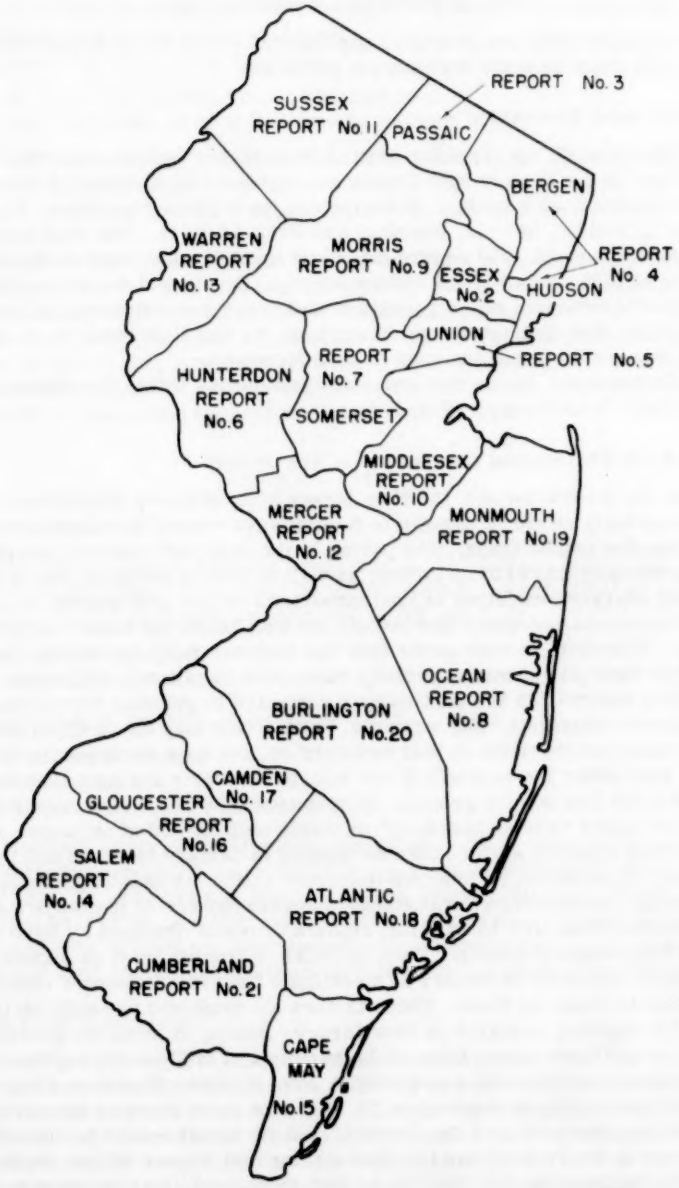


Fig. 1 - B Counties and County Reports of the Soil Survey.

4. to report on the major soil types and some of their properties as found and tested in the glaciated Newark metropolitan area, and

5. to illustrate some practical applications of the newly prepared engineering soil maps to some engineering problems.

Area under Discussion

The Newark metropolitan area of New Jersey is here somewhat arbitrarily defined as the area in New Jersey encompassed by a radius of about 20 miles from Newark as a center. It comprises the following counties: Bergen, Essex, Hudson, Morris, Passiac, and Union (Fig. 1). The total area of these counties is about 1232 square miles, or approximately one-sixth of the total area of 7836 square miles of New Jersey. Because of the extensive network of traffic arteries, dense population and heavy concentrations of business and industry, this district around Newark, N. J., and New York, N. Y. is called the Newark metropolitan area of New Jersey.

Geologically, the region under discussion lies within the glaciated area of Northern New Jersey. (Fig. 1-A).

Need for Engineering Soil Survey in New Jersey

In the middle forties, the New Jersey State Highway Department became increasingly aware of damage to its highways caused by climate and the ever-increasing traffic loads. The paving of the relatively narrow and yet expensive highway and railway ribbon, as well as that of airports, has a comparatively shallow foundation as contrasted with larger and heavier structures, the foundations of which are usually set well below the frost penetration depth line. This fact, as well as the fact that they are fully exposed to the weather, makes their pavements especially vulnerable to climatic influences. Soils used in earthworks are particularly subjected to periodic temperature and moisture variations. For example, frost action may cause differential heaves on roads, but variation in soil moisture content as a consequence of frost action may affect the strength of the soil, particularly during a thawing period after frost has left the ground. This causes loss of bearing capacity of soil and so-called "spring break-up" on roads built on and of improper soil. The resulting effect of such conditions usually is damage to roads and their pavements. Thus it can be inferred that frost action not only imposes difficulties in design, construction, exploitation and maintenance of highways but also impairs traffic safety. In addition, repairs to roads damaged by frost usually cost huge sums of money. Thus, in order to combat frost on highways effectively, it becomes necessary to investigate the various factors contributing to damage to roads by frost. These factors all combined to establish the necessity for highway research in New Jersey. Having in mind the general problems as outlined above, Rutgers University and the New Jersey State Highway Department established a cooperative Joint Highway Research Project, and eleven years ago, on September 12, 1946, the Joint Highway Research Committee, administrator of the Project, held its initial meeting. One of the objectives of the Project was the Engineering Soil Survey of New Jersey.

The Engineering Soil Survey, as just mentioned, may be considered as a means of (1) taking inventory, following the principle that one has first to know what types of soil are available in the state and what is their distribution, and (2) providing applicable data on the engineering characteristics of soils for

design, particularly preliminary design, and construction purposes of roads and other earthworks. Thus, the Engineering Soil Survey was planned to comprehend the entire State of New Jersey.

The engineering soil survey required

a) that the soils be identified and classified, and
b) that the boundaries of each soil type be defined.

The work was executed in the Soil Mechanics and Foundation Engineering Laboratory at Rutgers University. The resulting engineering soil maps (1) provide a graphical record of the fulfillment of both of the requirements described.

New Jersey is the first state in the Union to have a complete state-wide engineering soil survey with soil maps available, and informative material in the form of report-bulletins published. These bulletins are a valuable reference source for obtaining quick information about the general engineering properties of the soils of a particular area. The Engineering Soil Survey contains, thus, descriptions of soils in the Newark metropolitan area of New Jersey, which is the actual topic of this article (2, 3, 4, 5, 6, 7).

Geological Notes

Physical Features

Although New Jersey is small in areal extent, its geology, physiology and soil materials are rather complex. By nature New Jersey is situated within parts of two distinct geographic provinces, which are

1. the Coastal Plain and
2. the Appalachian Province.

The Appalachian, in its turn, is subdivided into

a) the Piedmont Plateau,
b) the Highlands, and
c) the Appalachian Valley and Ridge (Fig. 1).

The southwestern parts of the Piedmont and Highland are unglaciated, whereas their northeastern parts, as well as the Appalachian Ridge and Valley, are glaciated (Fig. 1). The soils in Bergen, Essex, Hudson, Passaic, Union, and the northern part of Morris Counties under discussion thus are situated in the glaciated part of northern New Jersey. This implies that in the Newark metropolitan area of New Jersey there are several varieties of soil types.

Principal Rock Formations

Northern New Jersey has different geologic formations which extend in a general northeast-southwest direction. The variety and distribution of the principal consolidated or solid rock formations in the glaciated area of New Jersey and in the adjacent part of New York is shown after Salisbury(8) in Fig. 2. These rocks contribute the main components for the soil in the respective areas and influence the land forms. The glaciated Piedmont area (Essex, Hudson, and parts of Bergen, Morris, Passaic, and Union Counties) is underlain mostly by sedimentary rock formations such as schists, conglomerates, sandstone, Brunswick shale, and igneous rock intrusions like gneisses

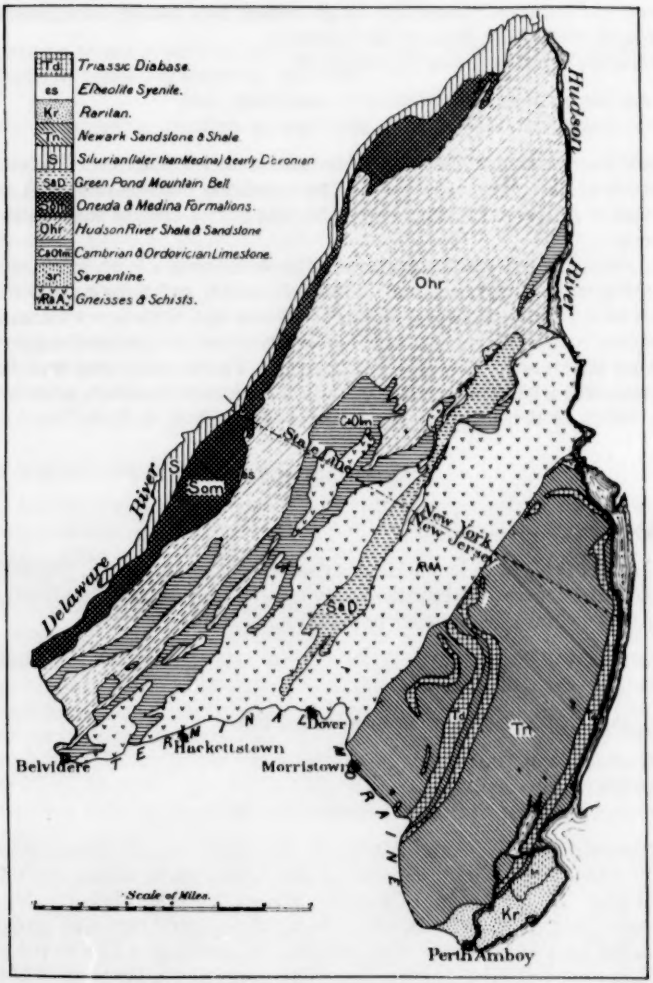


FIG. 2. Map Showing Underlying Rock Formations in the Newark Metropolitan Area of New Jersey.

and diabase. The diabase forms a prominent ridge, namely the Palisades. The extrusive basalt forms the well-known Watchung Mountains.(9,10) The sedimentary rocks give an undulating land form. North of the terminal moraine the land form producing rocks are covered by glacial deposits, thus masking out the rock surfaces. The rock-forming plains, hills, ridges, and mountains, it is understood, antedated the glacial deposits or the glacial drift.

The glaciated Highland area (parts of Bergen, Morris, and Passaic Counties) is underlain by gneiss, conglomerate, limestone, sandstone and shale. These rock formations, overlaid by glacial deposits, extend from the New York State line through the terminal moraine line southward to a line through Trenton and New Brunswick. In the majority of places the solid rocks are covered with a layer of unconsolidated material which could be

1. the result of decay of the underlying rock, or
2. the drift (unconsolidated material) deposited on the solid rock by glaciers.

Glacial Time

Geologically, we are living now in the Quaternary period. This period, in its turn, is divided into

1. Pleistocene epoch or glacial time and
2. recent epoch, or the time since the last withdrawal of the ice sheet. The geologic formations of recent alluvium are found along many river courses and as swamp deposits and beach deposits.

Late in the Pleistocene epoch, approximately some 30,000 or 40,000 years ago, about half of the continent of North America, and also part of northern New Jersey, were covered with a sheet of continental ice, termed the continental glacier. The thickness of the ice sheet varied between several hundred to several thousands of feet. The ice sheet was thick enough to fill up river valleys and override mountains of medium height. It modified the topography somewhat and left the already mentioned extensive deposits of so-called glacial drift. "Glacial drift" is the general term used to designate all of the superficial material of rock debris of any sort handled in any way by the continental glacier (erosion, transportation, deposition from ice or running meltwaters emanating from the ice). It is a term for glacial deposits of all types. The drift material includes till, stratified drift, and scattered rock fragments. It may be heterogenous, or it may be sorted, assorted, or stratified. The material consists of boulders, gravel, sand, silt, and clay in various gradations and proportions.

According to Salisbury, at least three ice advances or glacial stages are recorded in the drift deposits now existing in New Jersey. The glacial stages alternated with warmer interglacial periods. The continental ice sheets, after melting, retreated north far beyond the boundaries of New Jersey. In New Jersey, beginning with the most recent, the following three glacial stages are recognized:

1. The Wisconsin glacial stage of late Quaternary time (these glacial deposits are called the Wisconsin drift from their great development in Wisconsin),
2. The Illinoian glacial stage, and
3. The Kansan or Jerseyan glacial stage (of great antiquity).

Because in the glaciated part of New Jersey the older Illinoian and Jerseyan drifts have practically nowhere been recognized underneath the Wisconsin drift, let us refrain from discussing them here. However, for the engineer, the most intimate interest lies in the youngest glacial stage, namely in its associated Wisconsin drift.

The Wisconsin Drift

The direction of the movement of the Wisconsin stage ice sheet, in general, was southward across northern New Jersey, diverging at the margin of the ice right and left as illustrated in Fig. 3.

In its southward movement the continental glacier buried fully or carried away all of whatever older drift material remained in the area covered by it. In the courses of its forward advance the glacier broke, eroded and removed from extensive areas the mantle of the decayed and disintegrated rock. Some of the eroded and broken material was transported or "drifted" by the ice to its present position from regions to the north of New Jersey, for example from the State of New York and from Canada. The fine material—silt and clay—transported by water in suspension, was sedimented in still waters, for instance, ponds, lakes and the sea.

The glacial drift is not restricted to any particular rock, but is found distributed over all the various rock formations in the glaciated part of New Jersey, and is independent of the underlying rock.

During the Wisconsin stage of the glacial epoch the southern margin or end of the ice sheet maintained a relatively constant position for a considerable length of time, according to glaciologist Antevs, for about 2000 years.(11)

The coarse materials of the drift--gravel and sand—were deposited principally along the glacier stream courses, and, because they are waterworn, they are more or less stratified.

Terminal Moraine

The southern extension of the continental glacier in New Jersey during the Wisconsin glacial stage is distinctively identified by the great terminal moraine (Figs. 1, 3 and 6) which crosses Staten Island, N. Y., New Jersey and Pennsylvania; chains of moraine hills extend to the west all the way across the North American continent.

A moraine is a complex accumulation of mixed glacial drift deposited by the direct action of glacier ice. In a terminal moraine there may be found beds of stratified gravel and sand, intermixed with unassorted boulders, gravel, sand, and fines. A moraine has its own topographic expression or land form. A moraine formed as a ridge or a row of low, irregular, knobby, hummocky hills at the extreme margin of the continental ice sheet is termed a terminal moraine. It is formed of a belt of ridges at right angles to the motion of the ice sheet, and was brought about where the melting of the glacier ice approximately equalled the rate of advance of the ice, i.e. the terminus of the ice sheet stood in approximately the same position for a relatively long period of time. Here, under such conditions, during this equilibrium position of the ice, all of the material that was frozen into the ice sheet was deposited, accumulating itself in great piles or ridges of glacial debris—the terminal moraine (see Figs. 4 and 5).

A moraine formed at the lines of ice in front of the retreating, or melting glacier leaving a series of approximately parallel terminal moraines (north of the terminal moraine) is called a recessional moraine.

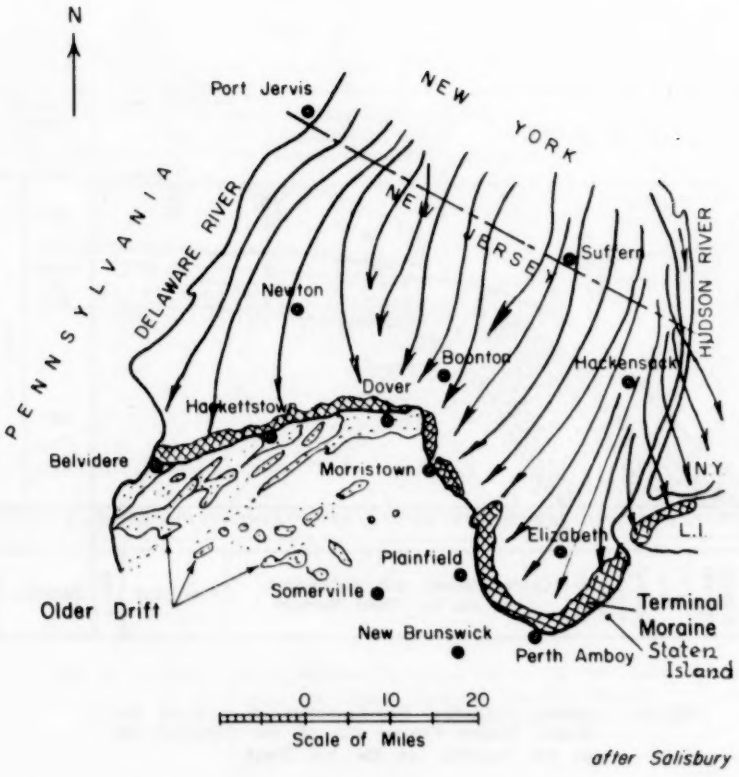


FIG. 3. Illustrating the Direction of Advancement of the Wisconsin Glacier, Terminal Moraine, and some Older Drift South of the Terminal Moraine.

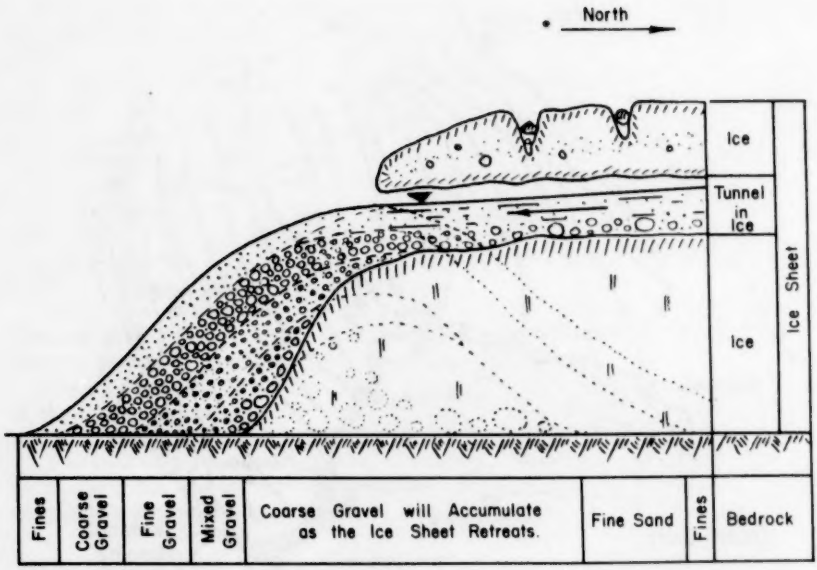


FIG. 4. Sketch Illustrating the Formation of a Kame by a Glacial Stream Flowing Out of the Tunnel in Ice at the Terminus of the Ice Sheet.

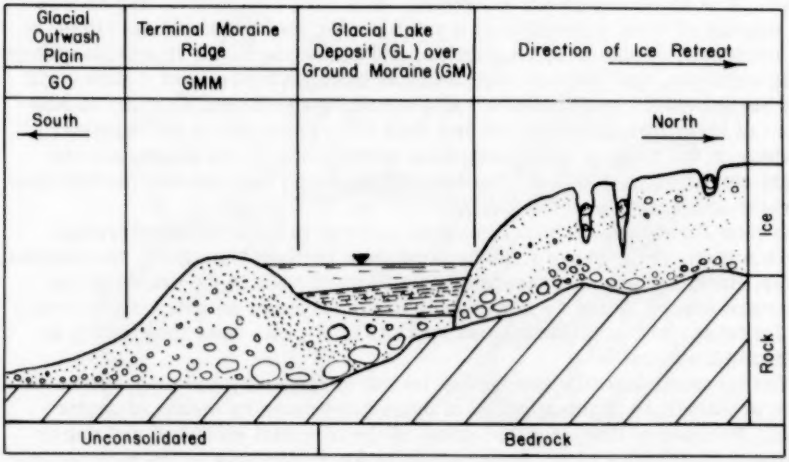


FIG. 5. Theoretical Illustration, in Cross Section, of the Relations of a Glacial Ice Sheet, Terminal Moraine, Ground Moraine, Glacial Lake Bed Deposit and Glacial Outwash Plain.

As a glacier melts away, it leaves a thin cover of unsorted glacial drift on the ground beneath the ice. Such a drift is called the ground moraine.

It can be said that the terminal moraine is fronted south of it by a glacial outwash plain, and backed north of it by a ground moraine (Figs. 4 and 5). Hence, a terminal moraine indicates the position of the former front of the continental glacial ice sheet.

As to the material, the terminal moraine can also consist of unsorted glacial till—a mixture of large and small rock fragments down to the clay sizes. It is loose and easily excavated. The rock composition represents the species of rock it overlies at a given place: red shale and sandstone in the Piedmont, gneiss in the Highlands, and limestone in the limestone valleys. At Morristown, for example, the terminal moraine consists of a clayey silt with some pebbles and boulders. At Netcong, the terminal moraine is composed of clean, stratified gravel and sand.(12) The width of the terminal moraine in the form of hummocky hills is from one to two miles, and the height can be about 100 feet. Between Madison and Morristown, the terminal moraine appears as a broad ridge.

In New Jersey the terminal moraine crosses in a curved line through Perth Amboy, from where it bends northward through Plainfield, Morristown, Dover, Hackettstown, and thence a little south of west to Belvidere on the Delaware River. About 23 miles south of the terminal moraine there occurs the Jerseyan drift in discontinuous spots (Fig. 6); this ends indefinitely in scattered boulders.

The terminal moraine, containing lenses of silty clay and clay, offers some problems as to the disposal of domestic wastes by means of septic tanks. Because of this, in some areas of the terminal moraine, and due to improper functioning of domestic septic tanks, unpleasant odors may become a nuisance to the whole community.

Successive terminal moraines formed during the retreat of the ice sheet, particularly the recessional ones, occasionally acted as earth dams. In such instances, the glacial debris of the meltwaters were deposited against them. Also, because of the uneveness of the moraine, such glacial dams obstructed valleys, blocked stream courses and closed depressions in the terrain, thus forming glacial lakes and swamps.

Newark "Meadows"

Among the noticeable geologic-topographic features in the Newark metropolitan area of New Jersey, mapped on the engineering soil map by the symbol F/MTM (fills in marine tidal marsh), is the so called Newark Meadows. It is situated west of the Palisadies Ridge and between Elizabeth and Hackensack, and is about 15 miles long and about four miles wide in the Rutherford-Newark area. The meadows, in the geologic past, are also thought to be a glacial lake occupying a great carving in the underlying sandstone and shale. The lake is now filled with very soft, compressible and stratified silty clay and clay. This lake bears the geologic term Lake Hackensack. There are two outcropping islands in the ancient lake, namely Laurel Hill, and Little Snake Hill. Both consist of intrusive diabase rock of the same origin as the Palisades. Laurel Hill affords an excellent view of the meadows in all directions, revealing impressively the industrial might and engineering structures such as bridges, roads, railways, waterways in the Newark metropolitan area of New Jersey—only a stone's throw from New York. Indirectly, the meadows and the structures, for example the foundations of the bridge piers of the New

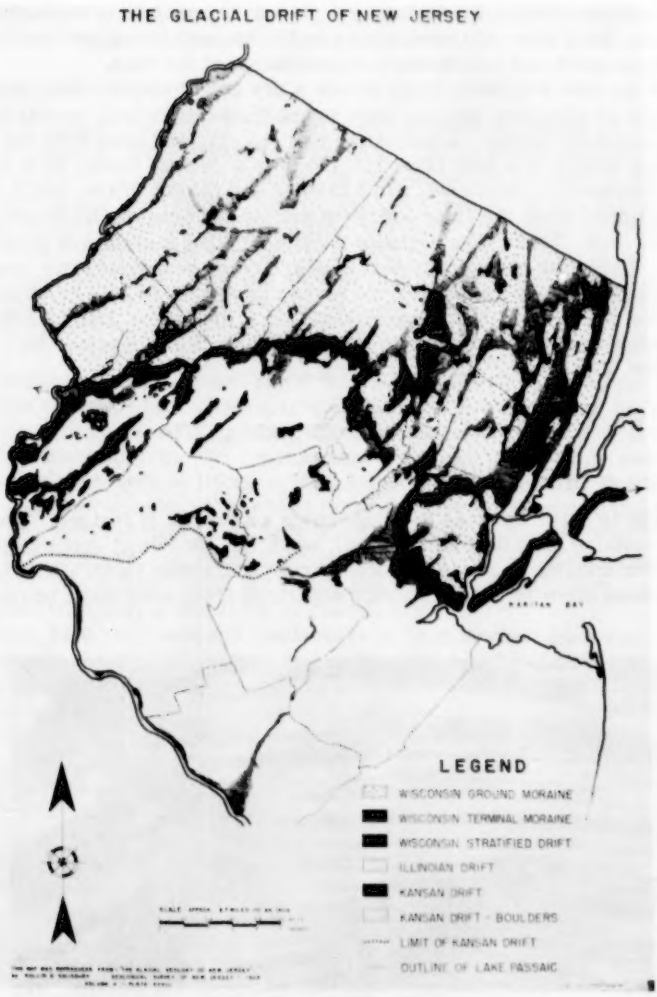


FIG. 6. Glacial Drift Map of New Jersey.

Jersey Turnpike, remind one of the design and construction difficulties with which the soils and foundation engineer had to cope: much water, little or no bearing capacity of soil, great depth, at places, to reach firm support for structures. Sand piles and sand drains had to be used to support the Turnpike and to attain quick and satisfactory consolidation of the fills.

Where the lake had been, there is now a dry bed of glacial clay. Sections of the walls of clay pits, viz. old lake, in the Hackensack area reveal neat annular layers or varves, an indication that the clay had once been the sediment at the bottom of a lake (Fig. 7). There is a thicker varve, light in color, which is the summer sediment, and a thinner and darker varve, which is the winter sediment when the finer sediment settled out beneath the frozen surface of the lake. These lake deposits show a striking sequence of layered deposits. Each pair of such summer-winter layers represents one year.

Glaciologist Antevs correlated the varves from lake to lake northward of New Jersey and arrived at the conclusion that it took about 2500 to 3000 years for the continental ice sheet to retreat from the terminal moraine to Haverstraw, New York.

The Glacial Drift

Constitution of Drift

The drift is heterogeneous in composition as well as in texture. Usually the drift contains huge boulders, gravel, sand, silt, and clay, sometimes intimately commingled in every possible variety and grade (unstratified), but at other times more or less distinctly separated from each other (stratified).

FIG. 7. Varved Clay of Glacial Origin from the Moonachie Area, Bergen County, New Jersey.

In mountainous areas the drift is coarse, manifested by the presence of boulders and other less coarse material, although clayey soils can also be found there. In the Hackensack Valley, for example, the glacial drift consists almost entirely of fine clay, underlain by a coarse drift.

Thickness of Drift

The thickness of the glacial drift covering the bedrock varies greatly within a short distance. This depends partly on the filling of the pre-glacial valleys, and partly on the variable amount of glacial debris present in different parts of the glacial ice. In New Jersey, the thickness of the glacial drift is estimated to vary from zero to more than 250 feet. A depth of 460 feet is an extreme known depth. This means that at some places the rock of one hill is hardly covered by a drift, whereas another hill is entirely composed of the drift. This should be of great interest and significance to the engineer. An average thickness varies approximately from 20 to 40 feet.

In soil and foundation engineering the heterogeneous, unconsolidated formation of glacial drift often necessitates determining the kind, thickness and depth of the soil material, as well as the depth and position of the bedrock. The practical importance of knowing the thickness of the drift is well illustrated by an example from the construction years ago of the large earth dam for the water reservoir at Boonton, New Jersey. The dam site originally selected has been abandoned; the hilly terrain where the dam was to be constructed was covered with loose drift unable to perform satisfactorily in respect to permeability, nor could it properly support the foundations of the dam. A new site farther up in the valley had to be selected where the bedrock was covered with only a thin layer of glacial drift.(8)

However, high railroads and roads were built in several places in northern New Jersey on the valley terraces and plains of the drift. Examples of railway engineering in the northern part of New Jersey are: along the Pequannock running in an east-westerly direction, and the Delaware, Lackawanna and Western Railroad, which is constructed on a thick drift which evens up the irregular surface of the underlying bedrock.

From what has been said one can infer that the thickness or depth of the glacial drift in a valley principally determines the construction expense of a dam, whether of earth or concrete, or a road, or railroad, as well as the cost of foundations for bridges or other heavy engineering structures. Nor should the glacial drift be forgotten as a means of combating the water problem in foundation and earthwork engineering.

Division of Till

The glacial drift deposits can be grouped into two principal types, namely:

1. the glacial till, and
2. the glacio-fluvial deposits.

1. Glacial Till

The till is found north of the terminal moraine practically everywhere not covered by stratified drift. Till is the unsorted, unstratified, unconsolidated, heterogeneous material deposited directly from the ice (ice has no sorting power), i.e., with little or no transportation by water. It consists of a great variety of fragments of rock in a clayey matrix, for example, boulders, gravel, sand and silt of various sizes and in various proportions. The wide range in

particle size is typical. The rock fragments are usually angular, but they may have flat surfaces where they were rubbed and ground against each other by the glacier. The till is sometimes found in a dense state, compacted by pressure, and therefore, in places, where the gravel has been cemented by limt carbonate the till can be excavated only by means of a power shovel, or even by blasting. Otherwise excavation in glacial till presents as a usual thing no problems except for the larger boulders, of which there may be plenty.

The permeability of the glacial till to water is a function of its clay content. Because of the erratic nature of the till in texture, its permeability is also erratic, and has to be checked in each case very carefully. Drainage and domestic sewage disposal are problems for consideration.

In places, the glacial till or unstratified drift may consist of clay-like rock flour, sometimes referred to also as glacial clay, containing admixtures of boulders, gravel, sand and silt. In the vicinity of Morristown, along the toes of the gneissic ridges, there are deposited layers of saturated gneissic rock flour, gray in color, about 7 to 10 feet below the adjacent ground surface. Local contractors call this soil material by an unscientific term, "the bull's liver." Contractors have difficulties in excavation and foundation work in this rock flour, which has a tendency to slump, collapsing unsupported walls as well as supported walls if the ground-water table is not lowered. Also, contractors fear losing construction machinery operating on the rock flour.

2. Glacio-Fluvial Deposits

These are deposits which have been worked over or sorted in layers by the meltwaters of the ice. These deposits are known as "stratified" or "modified" drift.

The glacio-fluvial deposits comprise

a) valley trains,
b) glacial outwash plains,
c) stratified deposits of coarse and fine material such as eskers, kames, and kame terraces, and stratified drift along the major drainage courses north of the terminal moraine, and
d) glacial clays, found in the Hackensack and Passaic valleys and in the swamp areas between the Watchung Mountains and the Highlands.

a) Valley Trains - Valley trains are narrow deposits of glacial gravel, making one of the conspicuous features of the glacial epoch south of the terminal moraine. Valley trains characterize some of the southward drainage courses, as, for example, the Delaware Valley.

b) Glacial Outwash Plains - Glacial deposits of stratified gravel and sand and some silt of the till which were eroded, washed out, reworked, sorted and redeposited by flowing waters from melting glacier beyond the southern boundaries of the ice sheet and found beyond or in front of the terminal moraine are called "glacial outwash plains." The rock fragments transported for a considerable distance by the flowing water are well rounded. There are, for example, glacial outwash plains northwest of Metuchen in the vicinity of Dunellen and Plainfield, consisting of gravel and sand washed out from the terminal moraine by the ice waters. This glacial outwash plain deposit contains enough shale and sandstone to give the deposit a reddish tinge. The degree of sorting of the outwash is variable. However, in general, there prevails a gradation from coarse to fine away from the moraine.

The smooth, sandy outwash plains, underlain by gravel, tend to have good drainage. Hence, such places are well suited for building airports, and for camp-sites, such as those established on the outwash plains south of the terminal moraine on Long Island, N. Y. Housing developments are also well accommodated on such formations.

c) Stratified Deposits - Under this category the following types of stratified drift are distinguished:

i) eskers,
ii) kames and kame terraces, and
iii) the so-called stratified drift along the major water courses north of the terminal moraine.

i) Eskers - Esker is a geological term to indicate a conspicuous, long, narrow, low, rather smooth-surfaced, sinuous ridge of crudely stratified glacio-fluvial gravel and sand. Such a land form is also known as the serpentine kame. An esker is a ground moraine, oriented approximately in the direction of movement of the glacier, or, in other words, roughly paralleling the direction of the ice movement. According to one theory, eskers are thought to be the fillings or debris of sub-glacial channel streams, crevasses

FIG 8. An esker from the top of which fines have been stripped. End view. Flat foreground has already been excavated.

and tunnels in stagnant or almost stagnant ice sheets which have been deposited during the times of progressive retreat of the ice front, appearing as ridges of gravel and sand (Fig. 8). The cause of the deposition is an obstruction in the ice tunnel. Another theory maintains that eskers are deposits of glacial debris accumulated in ditches and open stream channels in the surface of the ice. Upon melting of the supporting ice walls and the ice sheet itself, the debris is gradually lowered and finally slumps down, forming the characteristic esker ridges.

In New Jersey, eskers can be found in the Allendale-Ramsey area in Bergen County. An esker about three miles long is located in the Florham Park-Hanover area, in Morris County. There are also some eskers in Sussex and Union Counties.

ii) Kames and Kame Terraces - Kames are glacio-fluvial deposits. They appear as relatively low mounds, hills and short ridges consisting of poorly sorted and poorly stratified gravel and sand. Kames are diverse in size, and may be about 100 feet in height above the surrounding ground.

Because the margin of the ice sheet surely must have been of a complex character, it can be assumed that kames were formed by the pounding meltwater depositing its debris within, among and underneath the edges of the glacier along the lines of retreat of the continental ice sheet (Fig. 4). In New Jersey, kames are encountered in Bergen, Essex, Hudson, Middlesex, Morris, Passaic, Sussex and Warren Counties.

Kame terrace is a stratified drift formed between the ice sheet and adjacent higher ground, remaining as a terrace after the ice has disappeared.

iii) Stratified Drift Along the Major Drainage Courses - Under this category of glacial deposits are discussed the stratified drift along the major drainage courses north of the terminal moraine which are deposited by water flowing within or out from the continental ice sheet. Stratified parts of moraines are also comprehended within this category. The stratified drift contains relatively homogeneous sand with various percentages of gravel and silt. Gravel is found in layers of various thicknesses, and in varying extent.

Some glacial stratified drift deposits generally provide the best road construction material and railroad ballast. Some of these beds, having sharp and angular sand particles, as well as containing good quality coarse material, are very desirable aggregates for concrete. This particularly pertains to some well-washed kames. However, according to the author's observation kames and eskers containing good material, are being rapidly used up in New Jersey, or are becoming unavailable because they are very desirable land for housing and golf course developments.

d) Glacial Clays - The old basins are filled with stratified silty clays and clays. They are soft and very compressible. The sediments were formed by underwater sedimentation, and have never been subjected to more than their own submerged weight. Therefore these materials have low densities—about 40 to 70 pounds per cubic foot.(4) Fig. 7 shows a bed of varved clay in the Tri-State Brick Corporation's pit at Moonachie (south of Hackensack), in Bergen County. The varves of the clay are relatively thick, and the clay itself very erosive; in three days water flowing through a freshly opened channel carved out a three foot deep crevasse in the bank of the clay.

Borings put down in this clay deposit to 82', 85' and 92' depth did not reach bedrock.

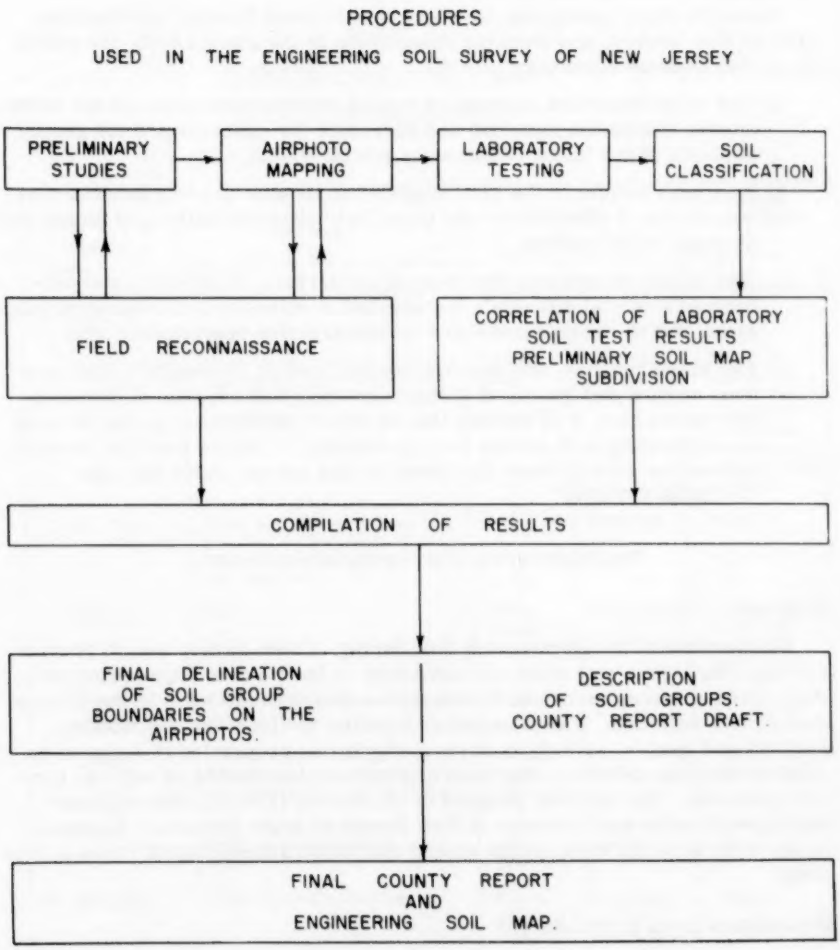

PROCEDURES
USED IN THE ENGINEERING SOIL SURVEY OF NEW JERSEY
PRELIMINARY STUDIES
AIRPHOTO MAPPING
LABORATORY TESTING
SOIL CLASSIFICATION
FIELD RECONNAISSANCE
CORRELATION OF LABORATORY SOIL TEST RESULTS PRELIMINARY SOIL MAP SUBDIVISION
COMPILATION OF RESULTS
FINAL DELINEATION OF SOIL GROUP BOUNDARIES ON THE AIRPHOTOS.
DESCRIPTION OF SOIL GROUPS. COUNTY REPORT DRAFT.
FINAL COUNTY REPORT AND ENGINEERING SOIL MAP.

FIG. 9. Diagram Illustrating the Procedures Used in the Engineering Soil Survey of New Jersey.

Summary on Glacial Drift Deposits

Table I is a summary of the principal glacial drift deposits in the Newark metropolitan area of New Jersey.

From the short geological sketch of the glaciated Newark metropolitan area of New Jersey, and from the description of the glacial drift, the following points become apparent:

1. The most important sources of soil as construction material for earthworks, foundation material and aggregate for concrete are the glacial deposits of the latest (Wisconsin) glacial epoch.
2. It is very helpful to the civil engineer to be able to recognize the various modes of glacial deposits from their characteristic land forms and geologic relationships.
3. The ability to appraise the mode of occurrence, application and performance of soil materials is essential to effective and intelligent judgment, and to reliable prediction of construction operations.
4. For these reasons, and because the engineer is interested in the location, amount and quality of suitable materials in advance of construction operations, it is obvious that an aid to the engineer in the form of an engineering soil survey is very desirable. Such a need for the soil survey has already been discussed in this article under the topic "INTRODUCTION."

The Engineering Soil Survey of New Jersey

Purpose

The purpose of the Engineering Soil Survey of New Jersey was to provide the engineer with a soil materials inventory in the form of engineering soil maps and to embody technical information relative to the soils in the form of descriptive bulletins. These bulletins describe the location, distribution, quantity and quality, as well as some geotechnical properties of soils as related to locating and relocating road alignments, and finding of suitable borrow material. The ultimate purpose of the survey is to help the engineer dealing with soils and highways in New Jersey to make important decisions in his work in order to facilitate design and construction of good roads in this state.

Procedures Used in Soil Survey

The engineering soil survey consists of the examination and classification of the soils in the field and laboratory, with special emphasis on their engineering applications, of mapping the soil types on the engineering soil maps, and of compiling and presenting the physical properties and other pertinent and essential data in a convenient reference source such as a handy booklet.

The procedures used in the engineering soil survey of New Jersey are illustrated by way of a diagram (Fig. 9).

The work started with preliminary studies, which consisted of reviewing all available geologic, pedologic and agronomic references, and aerial photographs. This material was evaluated, soil sample locations determined and, based on this work, field reconnaissance and air-photo mapping was begun.

Table I

SUMMARY

OF PRINCIPAL GLACIAL DRIFT DEPOSITS

IN THE NEWARK METROPOLITAN AREA

Drift Deposit	Land Form	Materials		Stratification of Drift
		Type	Sorting	
1	2	3	4	5
1. Terminal Moraine				
a) Coarse-Textured	Stony ridges parallel to the line of the ice front. Irregular skyline. Hummocky topography.	Gravel, sand, silt, some clay	Unsorted, or sorting very poor	None, or sorted material irregularly stratified. Intermixed, heterogeneous material.
b) Fine-textured	Silty and clayey ridges parallel to ice front line. Terrain relatively smooth and gullied.	Till	Unsorted	None
2. Outwash	Flat	Sand, gravel, some silt	Well sorted	Horizontal, with some bedding. Stratified.
3. Ground Moraine	Small hills with smooth, gentle slopes	Till	Unsorted	None
4. Eskers	Serpentine, winding, narrow and low ridges, approximately parallel to ice movement.	Gravel, sand, some silt	Poor to medium	Some stratification
5. Kames	Conical or irregular hills. Regular and irregular skyline.	Gravel, sand, some silt	Poor to medium	Some stratification
6. Lake Bed	Flat to gently rolling.	Silty-clay, clay, muck	Distinctively sedimented	Varves

Soil and non-soil boundaries as well as other pertinent information were marked on the airphotos. The mapping of soils is the delineation of their boundaries. In field reconnaissance work it is very difficult to trace the areal extent of a soil deposit because of man's inability to see great distances. Therefore the mapping of all counties was based on engineering interpretation of aerial photographs. The important idea in the application of aerial photographs in an engineering soil survey is that any given soil pattern will change if other soil material is deposited above the given soil pattern. This permits the classification of soil possessing similar physical properties and drainage conditions that influence performance. For example, the appearance of eskers, kames or lake bed on the aerial photograph is so characteristic that by means of their features the soil boundaries can be determined and delineated with relatively great precision. The evaluation of soil pattern and the interpretation of various factors discerned on the aerial photograph is an exercise in deductive reasoning.

In field reconnaissance work, soil samples were taken and forwarded to the Rutgers Soil Mechanics and Foundation Engineering Laboratory for testing. Details of soil testing are described in the bulletins of the engineering soil survey. The soils were then classified, the laboratory test results correlated, and those which fitted the preliminary map subdivisions established at the start were grouped and assigned a soil mapping unit symbol. This served to define the final soil group boundaries on the airphoto.

Where the geology and soil patterns and their changes on the aerial photographs turned out to be complex, additional field work was done to clarify the situation. After checking the work hitherto done, the compilation of the results began. Soil groups were finally arranged based on soil engineering properties, and the final delineation of soil boundaries on the airphotos was then made according to the definition of the soil mapping unit. The soil groups were described, the soil delineations were transferred from the aerial photographs to the county base maps, and thus the engineering soil maps were produced and the final county reports printed and published.

Because each of the aerial photographs used for soil mapping represented in itself an area of original research, and because soil mechanics methods were applied to this work, the resulting engineering soil survey may be considered as something unique, useful and new in the field of soil exploration and highway technology.

Soil Mapping Units in the Newark Metropolitan Area

The various soil-mapping units applied in the engineering soil survey to the Newark metropolitan area of New Jersey are summarized in Table II.

Some Physical Properties of Various Glacial Soils

Because in preliminary planning and design work it is necessary to have on hand some basic data on local soil conditions, such as texture, grading, plastic properties and compaction, such data were obtained by means of testing the various soil types in the Soil Mechanics Laboratory at Rutgers, The State University, and the test results reported in the engineering soil survey bulletins accompanying the engineering soil maps. Soil physical properties have a marked influence on earthwork and highway performance.

Table II

MAPPING UNITS USED IN THE ENGINEERING SOIL SURVEY
IN THE NEWARK METROPOLITAN AREA OF NEW JERSEY

Mapping Units	Counties: Bergen	Essex	Hudson	Morris	Passaic	Union	Brief Description of Mapping Symbols
AO				x	x		Stratified older alluvium.
AR	x	x	x	x	x	x	Recent alluvium.
F	x	x	x	x	x	x	Filled areas (man-made land).
GD-42		x					Glacial drumlins.
GE-12	x		x	x		x	Glacial eskers. The largest is located in Florham Park, Morris County.
GK-12	x	x	x	x	x	x	Glacial Kames. Individual hills.
GKF-12					x		Glacial Kames. Fields and group of small hills.
GKG-12		x			x		Glacial Kames. Group of hills.
GL-67				x		x	Glacial lake bed materials.
GM		x			x		Ground moraine.
GM-12					x		Ground moraine. Extra coarse till.
GM-24	x	x	x		x		Ground moraine. Desirable borrow material.
GM-4		x		x		x	Ground moraine, silty. Drainage poor to good.
GM-42		x			x		Ground moraine. Predominance of silt in the till.
GM-46	x	x	x		x	x	Poorly drained ground moraine. High percentage of fines.
GMC-46				x			Early drift of Jerseyan and Illinoian glacial stages, south of terminal moraine.

Table II
(continued)

Mapping Units	Counties: Bergen	Essex	Hudson	Morris	Passaic	Union	Brief Description of Mapping Symbols
MMg	x		x	x	x		Sandstone bedrock outcrops.
MMgC-46				x			
MTM	x	x	x			x	Marine and freshwater tidal marsh.
R	x	x	x	x	x		Variable and/or complex geologic, soil and cultural condition.
Ss				x	x		Quartzite and conglomerate forming Kittatinny Mountain. Also, Green Pond conglomerate.
Sc-46				x			Soil cover.
Sh		x		x	x	x	Bedrock outcrops of Triassic and Martinsburg shale.
Sh-2				x			Soil development on Martinsburg shale.
Sh-4				x			Red soil from weathered Brunswick shale.
Shl				x	x		Shaly limestone, covered with thin soil.
Sl				x			Limestones (Franklin, Kittatinny, Jacksonburg).
Sl-47				x			Soils derived from limestones.
Ss	x	x	x		x		Red Newark sandstone.
Ssh				x	x		Red sandstone and shale, gray Pequannoc shale, gray sandstone.
T-12					x		Glacial stratified drift of stream terraces.
TX-12					x		Same, of irregular shape.
Z	x	x	x	x	x	x	Swamps.
TOTALS	19	24	19	28	33	17	

Table II
(continued)

Mapping Units	Counties: Bergen	Essex	Hudson	Morris	Passaic	Union	Brief Description of Mapping Symbols
GMM-2½	x	x	x	x		x	Terminal and recessional moraine deposits.
GMM-½2		x					Terminal moraine with large silt content.
GMX-2½-					x		Ground moraine; deposits of variable texture. MX = exceptional area.
GMX-2½A					x		As above, but of complex geologic condition.
GO-½2		x					Stratified glacial outwash; granular. Near terminal moraine, or terrace deposits along streams.
GO-2½				x		x	Adjacent to the front of the terminal moraine. Large outwash plains.
GO-½						x	Silt phase of the outwash.
GS	x	x	x		x		Stratified glacial drift other than eskers and kames. North of terminal moraine.
GS-½2	x	x	x	x	x		
GS-2½	x	x	x	x	x	x	Large deposits; broad flood plain along the Pompton River.
GS-½	x		x		x	x	Prevailing silty texture.
GS-½2		x					Predominance of silt over gravel.
GS-½6	x	x	x		x		Poorly drained low areas.
GSR					x		
GSX					x		
GSX-½2					x		
GSX-2½					x		
lb	x	x	x	x	x		Basalt flows. Watchung, Long Hill, Hook Mountains; traprock.
lb-½				x		x	Thin soil cover over basalt rock.
lb-½6				x		x	Thick soil accumulations at bases of slopes and on broad upland regions.
ls	x		x				Diabase ridges; Cushetunk Mountains, Rocky Hill, Palisades.

Particle Size Integral Curves

Figs. 10 through 14 represent particle size integral curves for some glacial soils of New Jersey according to the U. S. Bureau of Soils Classification System. These curves indicate that the contents of fines vary considerably for each soil sample tested. The dotted pair of curves on each of these diagrams represent graphically A. Casagrande's criterion of frost susceptibility of soil. According to A. Casagrande,(13)

> "under natural freezing conditions and with sufficient water supply one should expect considerable ice segregation in non-uniform soils containing more than three percent of grains smaller than 0.02 mm, and in very uniform soils containing more than ten percent smaller than 0.02 mm. No ice segregation was observed in soils containing less than one percent of grains smaller than 0.02 mm."

For the evaluation of frost susceptibility of a soil this criterion has as wide an application in European highway engineering practice as it has in the United States. For a discussion of other frost criteria see reference (14).

According to Casagrande's criterion, and by Figs. 10-14, glacial eskers and kame materials seem to be the least frost-susceptible. Some of the glacial outwash and some of the stratified drift appear also to be less frost-susceptible. The rest of the glacial materials with high silt contents appear by this criterion to be frost-susceptible.

The integral curves represent on these graphs material from the A, B and C horizons. No general correlation relative to frost susceptibility of soils from the various horizons could be observed.

Maximum Dry Density of Some Glacial Soils

Fig. 15 shows a comparison between optimum moisture content and maximum standard dry density of some glacial soils. The plot shows that for each type of the glacial soils there is a maximum dry density at its optimum moisture content. The general trend of the maximum dry density with the increase in fines of the soil is also apparent. The glacial lake bed materials show the least densities as compared with other soil types mapped. This suggests that the stability of compacted earthwork made of this soil will rapidly decrease as compared with other glacial materials. The greatest densities appear in those soils having some silt and clay contents, for example, the ground moraines and the silty phases of the stratified drift. Also, it can be inferred from this figure that the coarser the soil particles, the lower the optimum moisture content. From this plot it is possible to judge the dry unit weight and shear strength of the material. It is often specified that such data be observed, along with plasticity data and information on soil permeability, in the construction of earth dams and highway and airport fills.

Although being of a qualitative and approximate nature only, mechanical analyses of soils are a valuable aid in evaluating the soil properties and soil performance in the field.

Flow Curves of Glacial Soil Materials

Figs. 16 and 17 represent flow curves of various glacial soils as obtained by means of the standard liquid limit tests. The flow curves are plotted on one-cycle logarithmic paper. One observes that generally, but not always, the

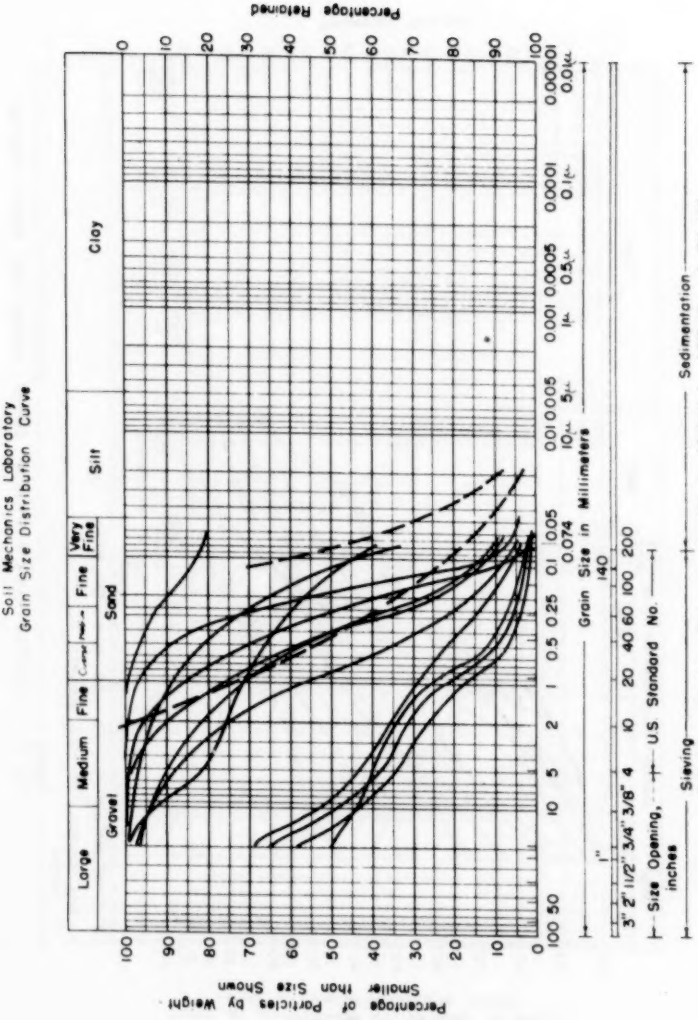


FIG 10. Glacial Esker and Kame Materials (GE-12 and GK-12)

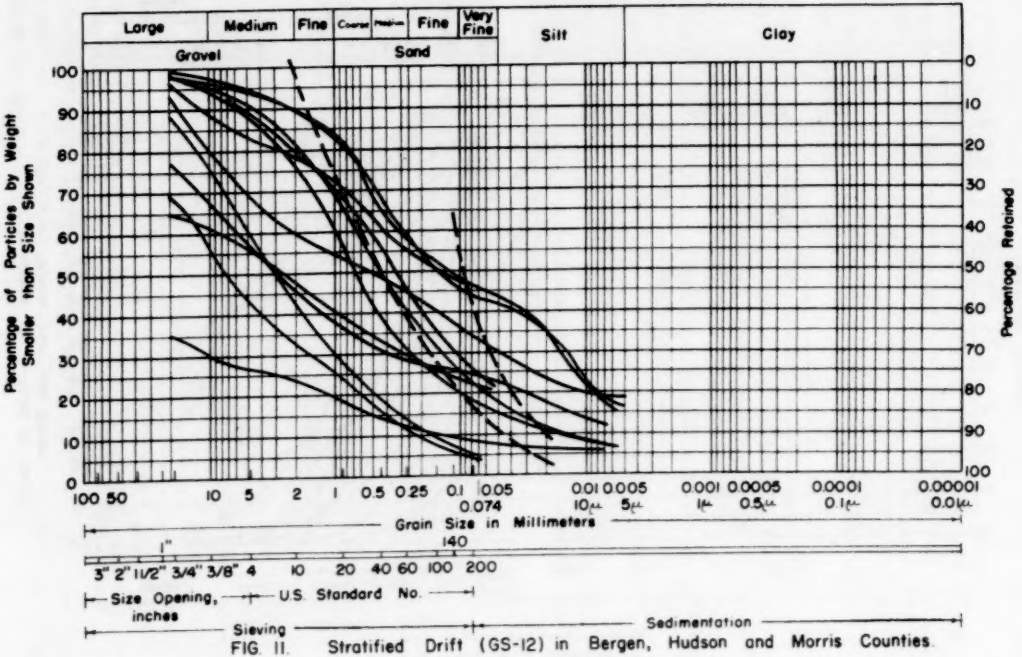


FIG. 11. Stratified Drift (GS-12) in Bergen, Hudson and Morris Counties.

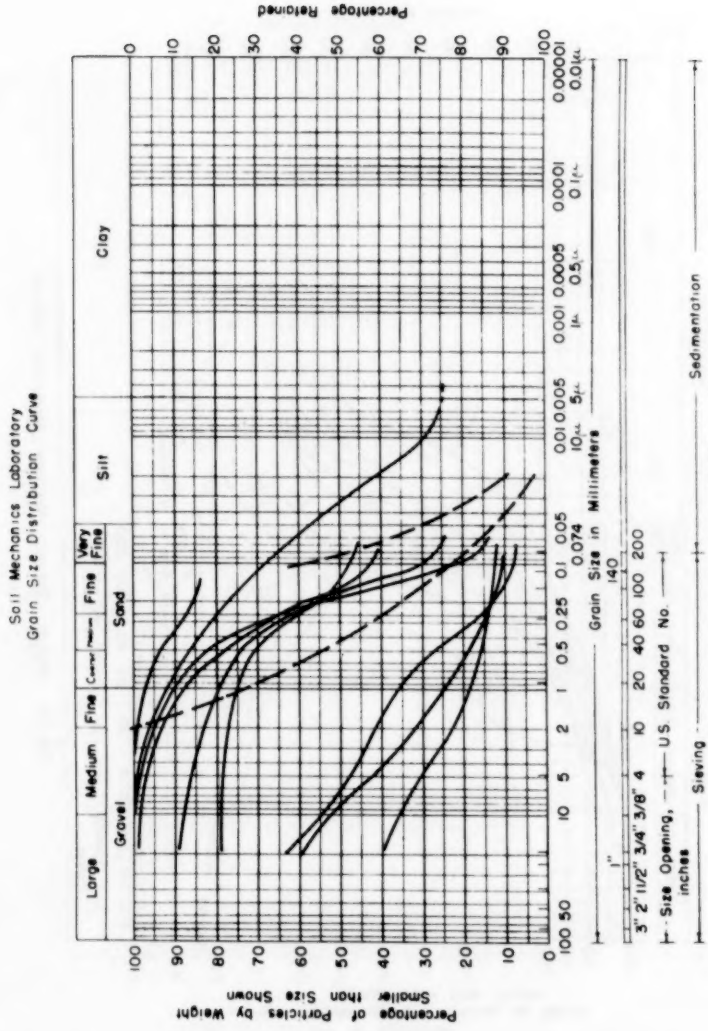


FIG. 12. Glacial Outwash Materials (GO-24)

Soil Mechanics Laboratory
Grain Size Distribution Curve

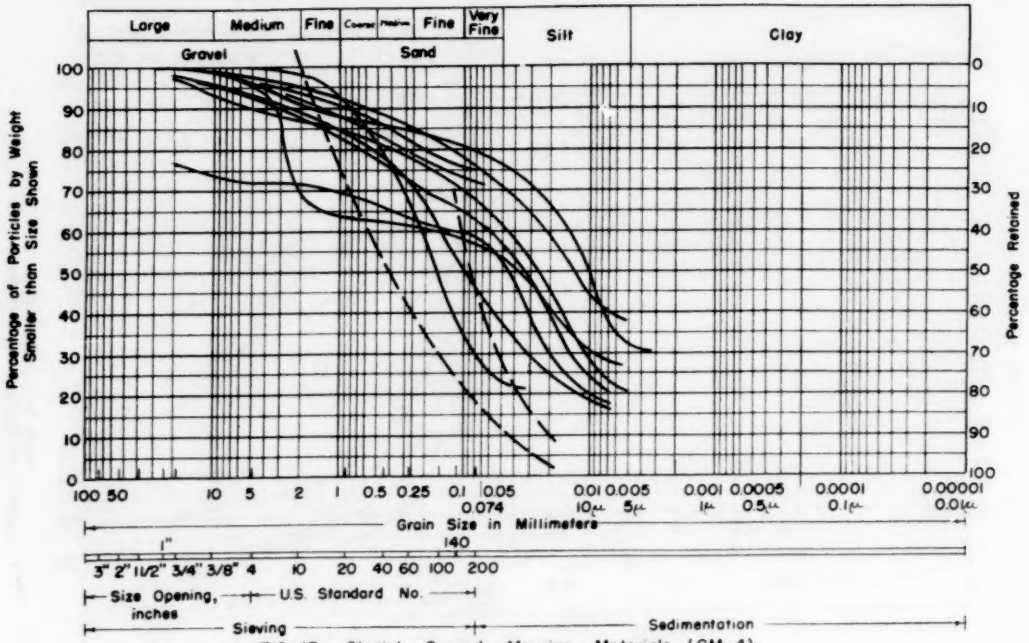


FIG. 13. Glacial Ground Moraine Materials (GM-4).

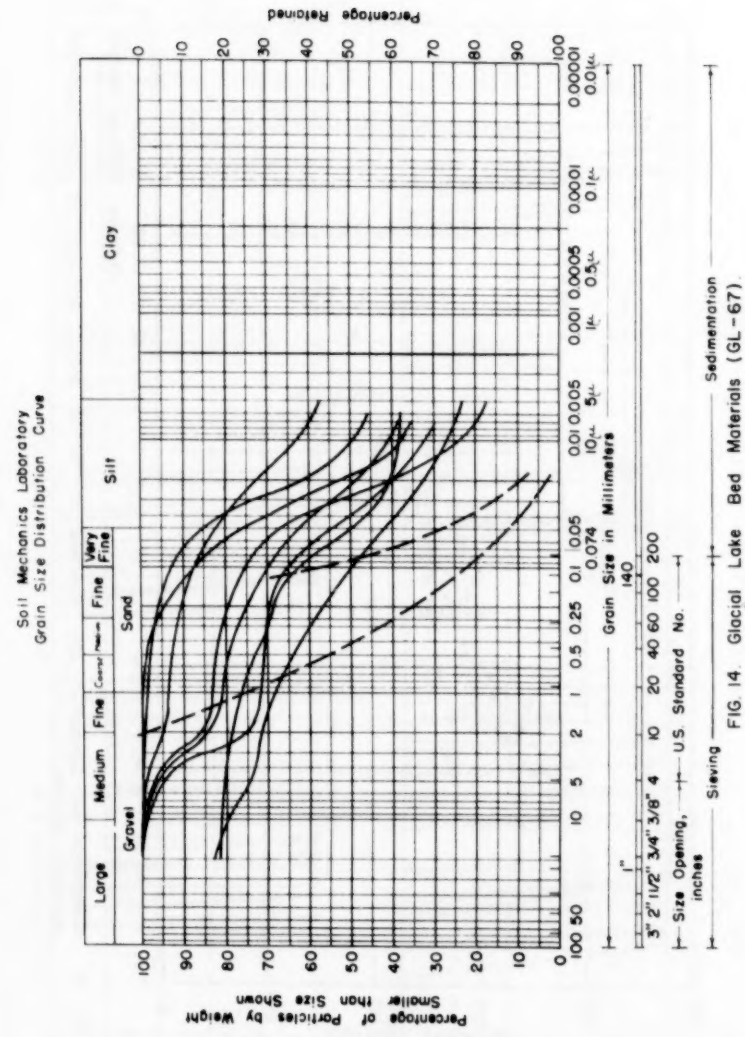


FIG. 14. Glacial Lake Bed Materials (GL-67).

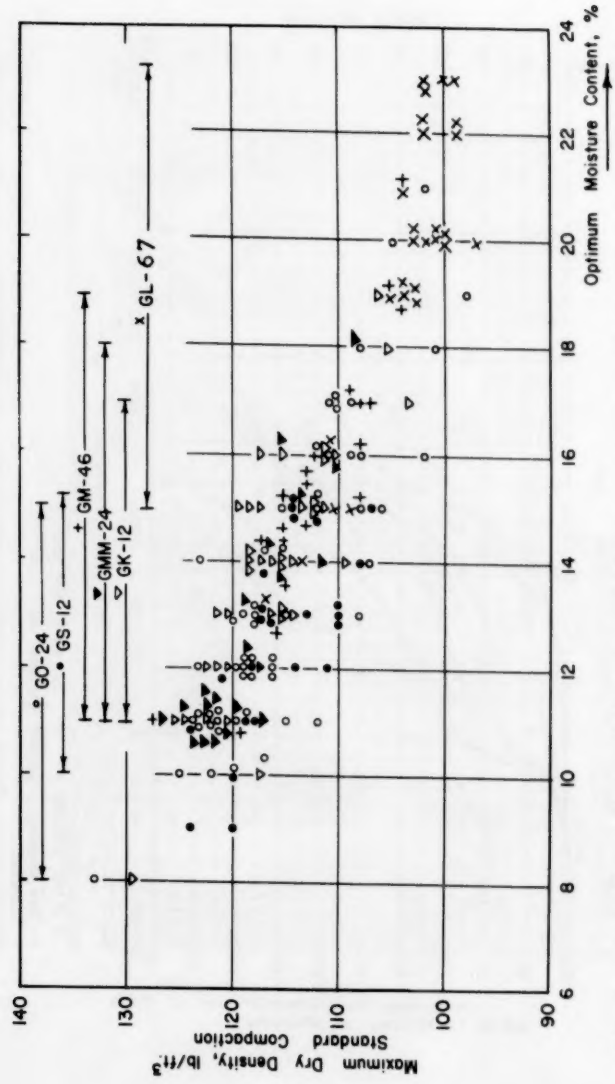


FIG. 15. Comparison of Compacted Dry Densities of Various Glacial Materials at Their Optimum Moisture Contents.

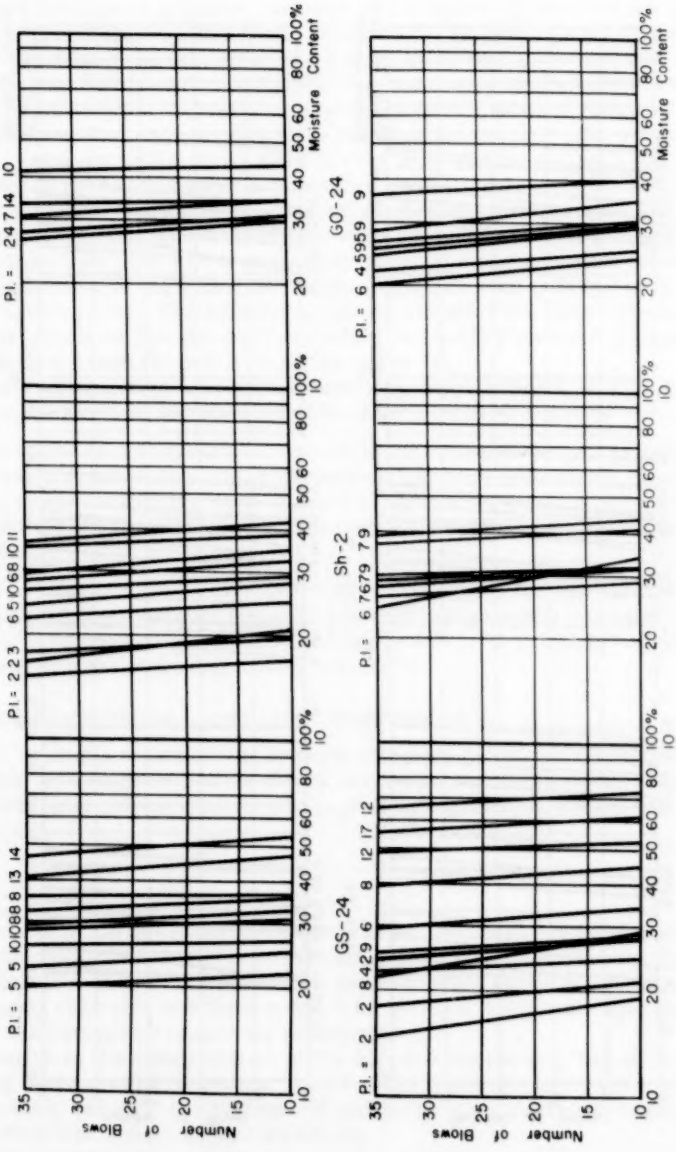


FIG. 16. Flow Curves of Glacial Materials

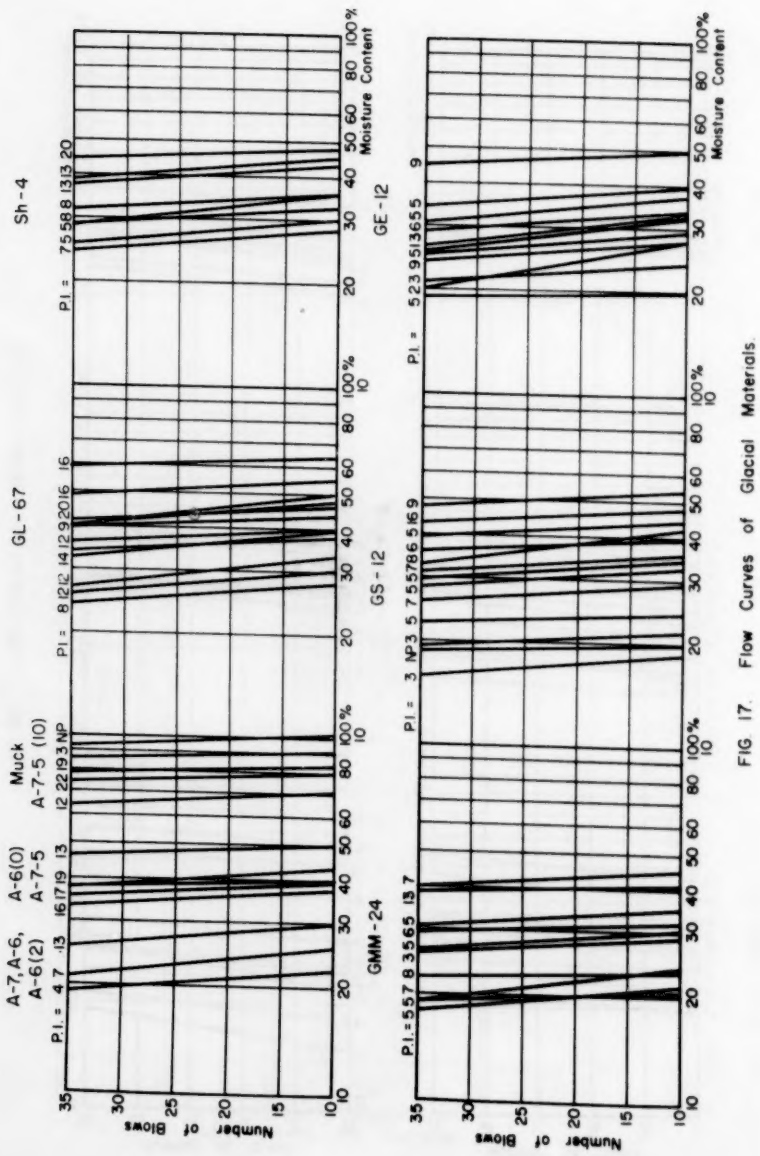


FIG 17. Flow Curves of Glacial Materials

plasticity indenx (P. I.) of soil increases as the amount of fines increases, and as the moisture content at the liquid limit increases. Here the heterogeneity and the silty-clayey character of soil is very clearly reflected. Note also that each type of glaciated material has a certain position in the optimum moisture cycle between 10 to 100 % of moisture, as well as a particular slope. The slopes, though, do not vary very much among themselves; however, there is difference enough in the various slopes to distinguish between the various degrees of cohesiveness and correlative shear strength of the various soils. For example, consider two flow curves, one with a flat slope (Curve 1) and one with a steep slope (Curve 2) (Fig. 18). Curve 1 and Curve 2 represent here, for the purpose of elucidating the point, two different soils but with the same P. I. - values. In this method of presentation it can be seen that in order to decrease the moisture content Δ m in both soils by the same amount, the soil with a steep flow curve (2) takes a greater number of blows (n_s) than that (1) with a flat slope, which requires only (n_f) number of blows; here $n_s > n_f$. The comparison of the slopes of the flow curves, thus, gives an indication that the soil with a flow curve (2) possessed a greater shear strength than the soil with a flow curve (1).

In describing his research on consistency limits of soils, A. Casagrande(15) explains the physical significance of the flow curves as follows:

> "The force resisting the deformation of the sides of the groove made in the soil pat is the shearing resistance of the soil. Therefore, the number of blows or shocks required to close the groove of a soil paste represents a relative measure of the shearing resistance of this soil at this moisture content."

The consistency limits and plasticity indexes given for the various soils in the engineering soil survey bulletins are thus valuable soil constants characterizing the plastic properties of those soils and their probable behaviour under varying moisture and loading conditions.

Optimum Moisture and Liquid Limit Relationships

Fig. 19 shows relationships between optimum moisture content and liquid limit for various glacial soils of New Jersey having various plasticity indexes. Such relationships are time savers in performing many standard soil compaction tests.

- - - -

Because of the great variety of soils in the glaciated area under discussion, and because soil may have different properties from place to place, it is advisable that soils for earthwork and foundation engineering purposes be thoroughly explored, and their properties tested in each particular case, as comprehensively and frequently as possible.

Relative to the interpretation of the soil test values as given in the engineering soil survey bulletins, it must be said that these test values should be interpreted, rationally evaluated and applied by each individual engineer in the light of his own personal experience.

In the interpretation of the test results, particularly with reference to the cohesive soils, a great deal, of course, depends upon the understanding of the materials which make up the clay fractions, and how they contribute to the properties of the composite soil.

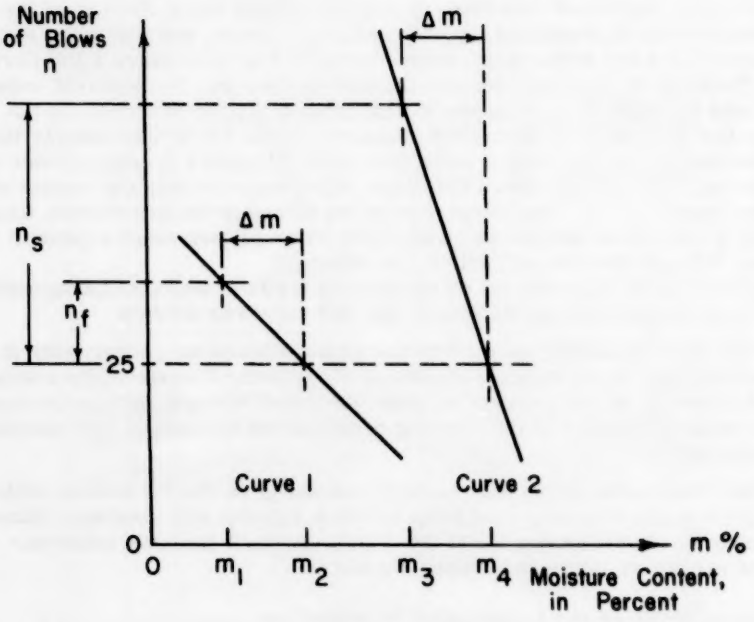


FIG. 18. Flow Curves

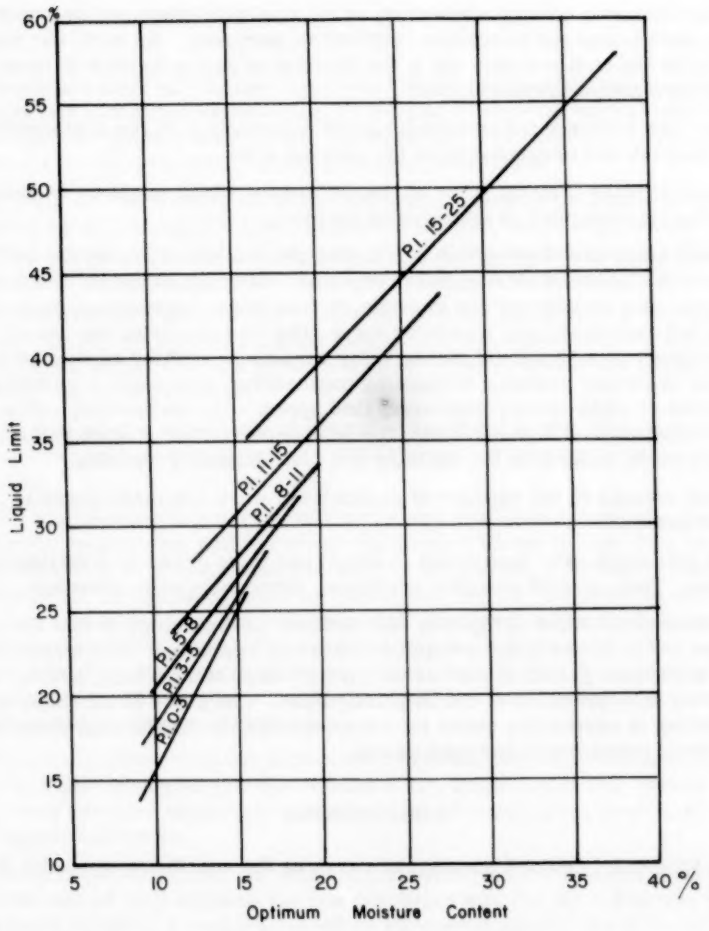


FIG. 19. Optimum Moisture - Liquid Limit Relationships of Various Glacial Materials for Different Plasticity Indexes.

Value of Engineering Soil Maps

The advantages of the engineering soil maps are manifold.

1. They contain a complete inventory of all available construction materials for earthworks and foundation engineering purposes. As such, the maps provide the best possible aid in the location of soil materials to meet certain prescribed specifications.
2. They are a source for obtaining rapidly information on the engineering properties and performance of the various soils.
3. The soil maps indicate some of the difficulties which might be encountered during the execution of engineering projects.
4. These maps are of great value in initial reconnaissance work for the location and relocation of alignments of roads, railroads, airports and in other engineering endeavors; for example, in foundation engineering, in preparing and executing time schedules for earthworks requiring the use of heavy equipment on soft ground during rainy seasons (= shifting equipment for work on firmer ground); in planning engineering, municipal engineering, studies of right-of-way problems, land appraisals, preliminary estimates of earthwork and highway costs; for road performance studies and in geologic work, as well as for military and civil defense purposes.
5. Great savings in the expense of preliminary work are made possible for contractors.
6. The soil maps have been found in actual use to be drawn to a satisfactory scale. They are of practical precision, dependable and convenient.
7. Another great asset to highway and earthworks technology is that the soil maps and bulletins make available to everybody geologic information and the knowledge gained in the survey through laboratory testing and the engineering interpretation of aerial photographs. The price of the maps and bulletins is reasonably small as compared with the actual total costs spent for their preparation and publication.

CONCLUSIONS

The following pertinent points may serve as the concluding remarks of this article.

Newark Metropolitan Area

1. Knowledge of the geologic features and location and distribution of land forms is very valuable in assessing the construction potentialities within the strategically located Newark metropolitan area.
2. The soil materials in this area were inventoried on an engineering basis by the bulletins and maps covering Bergen, Essex, Hudson, Morris, Passaic and Union Counties in the Engineering Soil Survey of New Jersey.
3. This region, lying within the glaciated area of northern New Jersey, contains much valuable land with fine construction materials, such as gravel, sand and silt.

4. According to the author's observation, good granular soil material is being rapidly used up, or becoming unavailable because it makes very desirable land for housing and golf course developments.

General

5. Stress has been laid on the importance of the engineering soil maps in soils and foundation engineering, as well as in highway engineering technology. In building earthworks, the soils with their physical properties should not be divorced from their natural land forms.

6. In using soil as an engineering construction material by means of which and upon which engineers build their structures, sound engineering practice requires that good use be made of all of the available scientific and technical knowledge in order to achieve safe and economical results.

7. The Engineering Soil Survey of New Jersey is a rapid source of reference, giving the engineer a good insight into the types, location and distribution, properties and performance of the soils in the Newark metropolitan area of New Jersey.

8. The descriptive bulletins accompanying the engineering soil maps translate the geologic and agronomic soil information into engineering language.

9. The Engineering Soil Survey of New Jersey has already served as a model for the engineering soil survey of the State of Rhode Island. The State of Delaware is at present also following in the footsteps of New Jersey. The Public Roads Administration in Washington, D. C., uses New Jersey survey methods to train some of their staff members in the field of soil surveys. Princeton University, Rutgers, The State University, New Jersey, and Brooklyn Polytechnic Institute use the New Jersey Engineering Soil Bulletins in some of their Laboratory and field work within the scope of their soil mechanics courses.

10. Originally meant only for highway engineering purposes, the past five years have revealed that the results of the Engineering Soil Survey of New Jersey are also applicable to many fields of engineering other than highways and airports.

11. From comments about the soil maps from engineers engaged in various branches of civil engineering one concludes that the Joint Highway Research Project, a cooperative effort between Rutgers, The State University, and the New Jersey State Highway Department, contributed greatly to the advancement of soil engineering and highway technology with the recently completed Engineering Soil Survey of New Jersey. The scope of the accomplished work can really be characterized as a "once-in-a-lifetime project."

ACKNOWLEDGMENT

The author expresses his sincere appreciation to Dr. E. C. Easton, Dean, College of Engineering, and to Professor J. J. Slade, Jr., Director Bureau of Engineering Research, both at Rutgers, The State University, New Jersey, for providing the means necessary for the preparation of this article.

The Wisconsin drift map was prepared after Salisbury by Mr. W. W. Holman.

The ground photographs are by A. A. Jumikis.

The author is also appreciative for the help given by Mrs. Ruth Ahrens, B.A., editor, in the editing and preparation of the typescript of this article.

Thanks are due to Mr. S. A. Johnson, of Moran, Proctor, Mueser & Rutledge, consulting engineers, New York, for his interest in this article, as well as for his suggestions and encouragement.

REFERENCES

1. Practical Applications of Engineering Soil Maps (Engineering Soil Survey of New Jersey, Report No. 22), by W. W. Holman, R. K. McCormack, J. P. Minard and A. R. Jumikis. Rutgers University Press, New Brunswick, N. J., 1957.

   Engineering Soil Survey of New Jersey, Rutgers, The State University, College of Engineering. Engineering Research series. New Brunswick, N. J.

2. Report No. 2 (Engineering Research Bulletin No. 16), Essex County, by F. C. Rogers, D. R. Lueder, G. H. Obear and W. H. Zimpfer, January, 1951.

3. Report No. 3 (Engineering Research Bulletin No. 17), Passaic County, by F. C. Rogers, D. R. Lueder and G. H. Obear, July, 1951.

4. Report No. 4 (Engineering Research Bulletin No. 18), Bergen and Hudson Counties, by D. R. Lueder, G. H. Obear, W. W. Holman and F. C. Rogers, September, 1952.

5. Report No. 5 (Engineering Research Bulletin No. 19), Union County, by F. C. Rogers, D. R. Lueder and Shangee Wang, September, 1952.

6. Report No. 9 (Engineering Research Bulletin No. 23), Morris County, by J. P. Minard, W. W. Holman and A. R. Jumikis, November, 1953.

7. Report No. 10 (Engineering Research Bulletin No. 24), Middlesex County, by W. W. Holman and A. R. Jumikis, November, 1953.

8. The Glacial Geology of New Jersey, by Rollin D. Salisbury, Vol. V, The Geological Survey of New Jersey, Trenton, N. J., 1902, p. 6.

9. The Geology of New Jersey, by J. V. Lewis and H. B. Kümmel, Geologic Series, Bulletin No. 50, State of New Jersey Department of Conservation and Economic Development, Trenton, N. J., 1940, p. 19.

10. The Physical Geography of New Jersey, by R. D. Salisbury, Geological Survey of New Jersey, Vo. IV, The John L. Murphy Publishing Co., Trenton, New Jersey, 1898, p. 104.

11. The Last Glaciation, by Ernst Antevs, Research Series No. 17, American Geographical Society, New York, 1928, p. 107.

12. Engineering Soil Survey of New Jersey, Report No. 1, Soil Environment and Methods of Research (Engineering Research Bulletin No. 15), by F. C. Rogers, Rutgers, The State University, College of Engineering, New Brunswick, N. J., revised edition, 1955, p. 27.

13. Discussion on Frost Heaving, by A. Casagrande, Proceedings of the Highway Research Board, Part I, Washington, D. C., 1932, p. 169.

14. The Frost Penetration Problem in Highway Engineering, by A. R. Jumikis, Rutgers University Press, New Brunswick, N. J., 1955, pp. 139-144.

15. "Research on the Atterberg Limits of Soils" by A. Casagrande, Public Roads, Vol. 13, No. 8, Washington, D. C., October, 1932, pp. 121-130; p. 123.

Paper 1647 May, 1958 SM 2

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

DEVELOPMENT OF MULTIPLE-WHEEL CBR DESIGN CRITERIA[a]

C. R. Foster,[1] M. ASCE and R. G. Ahlvin,[2] M. ASCE
(Proc. Paper 1647)

SUMMARY

This paper presents an improved method for developing multiple-wheel CBR design criteria from proven single-wheel criteria. It tells briefly of the development of the method and explains how it may be applied.

The development of aircraft landing gear assemblies using multiple wheels in various configurations created a need for airfield design criteria for this type of loading. Neither time nor economic considerations permitted the direct development of such criteria, and it was necessary therefore to relate the new loadings to existing, well-validated single-wheel-load criteria. Boyd and Foster presented a logical semirational method of relating multiple-wheel loadings to single-wheel loadings for purposes of design of flexible pavements in the CBR symposium published in 1950.(1) The method assumes that near the surface of the pavement the effect of a multiple-wheel assembly is the same as that of one of its individual wheels; while at appreciable depths, the effect of the assembly is the same as though the total assembly load was on a single wheel. Between these two extremes, the effect of an assembly is assumed to vary in a regular fashion from one to the other.

This method was used to produce multiple-wheel design criteria from established single-wheel criteria, and as soon as the multiple-wheel criteria were completed accelerated traffic tests to verify the development were begun

Note: Discussion open until October 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1647 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 2, May, 1958.

a. Paper presented at a meeting of the ASCE, Portland, Ore., May, 1958.

1. Engr., Chief, Flexible Pavement Branch, Soils Div., U. S. Army Engr. Waterways Experiment Station, Corps of Engrs., Vicksburg, Miss.
2. Engr., Chief, Special Projects Section, Flexible Pavement Branch, Soils Div., U. S. Army Engr. Waterways Experiment Station, Corps of Engrs., Vicksburg, Miss.

at the Waterways Experiment Station. Test sections were constructed and subjected to 70- and 100-kip twin-wheel loadings and 150- and 200-kip twin-tandem wheel loadings.

Results of these tests indicated that the design criteria first developed for multiple-wheels were slightly unconservative, thus precipitating a need for more realistic design criteria, or for a better means of resolving established single-wheel design relations into criteria for multiple-wheel loadings.

Accordingly, a complete review of all pertinent data was undertaken by engineers of the Waterways Experiment Station. Tests conducted at Marietta, Georgia,(3) and Stockton, California,(2) were re-examined, pertinent theoretical developments were studied(4,5,6) and the results of the multiple-wheel test section study were carefully reviewed.(7) This reanalysis resulted in a revised method for developing multiple-wheel design criteria.

In the following paragraphs, the essential elements of reasoning leading to the new method and selected data showing its validity are presented.

Pavement failure is associated with deformation or deflection and it appears reasonable that loadings producing equal deflections might well be considered to be equivalent. Yet it is known that a deflection or deformation which is tolerable when occurring over a wide span can be intolerable when occurring over a narrow span. This points directly to the need for examining the change in deflection with offset as well as the maximum deflection.

Thus, if it can be shown that a multiple-wheel load, which produces a maximum deflection equal to that of a certain single-wheel load, yields deflection versus offset curves at various depths whose slopes are less than those for the single load at equal depths, it may be concluded that the multiple-wheel assembly is creating no more severe loading than the single wheel. Pertinent data are available from stress distribution studies conducted by the Corps of Engineers.(4,5,6) Fig. 1 (theoretical developments) and Figs. 2 and 3 (results of tests on homogeneous clayey-silt and sand stress distribution test sections) show the relation between deflection versus offset curves at equal depths for single and dual assemblies. Without exception, the slopes of the deflection versus offset curves for the single loads are equal to or steeper than those for the dual loads at equal depths.

This analysis then shows that a single-wheel load that yields the same maximum deflection as a multiple-wheel load will produce equal or more severe strains in the subgrade or base than will the multiple-wheel load. The single load may, therefore, be considered equivalent to the multiple-wheel load for purposes of design. This equivalent single-wheel load and the existing, well-validated single-wheel CBR design curves provide a basis for developing designs for multiple-wheel assemblies.

Comparison of the theoretical curves of Fig. 1 with the test data curves of Figs. 2 and 3 shows that the theoretical curves are similar in general form to those derived from test data, and that for all but the shallowest depths the similarity of the curves is quite close. At the shallow depths discrepancies occur for the wide offsets, but at these depths, the maximum deflections for a multiple assembly are very largely the result of the load on one wheel. For this reason, discrepancies at wide offsets can have only a small effect, and it is therefore considered that theoretical deflections can be used in arriving at the relation between single- and multiple-wheel assembly loads.

Thus it has been established that a single-wheel load that produces the same deflection at a given depth as a certain multiple-wheel load is an equivalent single-wheel load. It has been further established that theoretical

deflections can be used to show this equivalency. It is now necessary to explain just how this equivalent single-wheel load can be determined.

Each wheel of a multiple-wheel assembly contributes a part of the maximum deflection occurring at any depth beneath such an assembly. Curves of deflection versus offset for various depths for a single wheel can be determined theoretically. Reference (4) includes a set of theoretical curves from which deflections at any offset and depth can be interpolated. Curves of deflection versus offset can be developed for multiple-wheel assemblies from the single-wheel curves by use of the principle of superposition. Now having single- and multiple-wheel curves of this type, the maximum deflections at a given depth beneath single- and multiple-wheel assemblies can be determined, and a relation between multiple-wheel and equivalent single-wheel loads can be established by equating these deflections. In equating the deflections, the contact area of the single wheel is taken to be constant and the same as that of one wheel of the multiple assembly.

By determining the equivalent single-wheel load for a number of depths throughout the pertinent depth range, a relation between depth and equivalent single-wheel load can be established. This relation can then be used to resolve the established single-wheel design criteria into criteria for multiple assemblies. A detailed example of the determination of equivalent single-wheel load is given in the appendix.

Design curves developed by means of the revised method are plotted in Fig. 3 as dashed lines along with the present design curves. In addition to these curves, Fig. 4 shows points indicating test section behavior under 2000 coverages of traffic. It is readily apparent from the service behavior analysis that the revised method of resolution of single- to multiple-wheel criteria is more in line with traffic data than the earlier method.

REFERENCES

1. Boyd, W. K., and Foster, C. R., "Design Curves for Very Heavy Multiple-Wheel Assemblies, Development of CBR Flexible Pavement Design Methods for Airfields, A Symposium." ASCE Transactions, vol. 115, p. 534 (1950).

2. Corps of Engineers, Sacramento District, Accelerated Traffic Test at Stockton Airfield, Stockton, California (Stockton Test No. 2). Sacramento, California, May 1948.

3. Corps of Engineers, Waterways Experiment Station, Certain Requirements for Flexible Pavement Design for B-29 Planes. Waterways Experiment Station, Vicksburg, Miss., August 1945.

4. _____, Investigations of Pressures and Deflections for Flexible Pavements, Report No. 1, Homogeneous Clayey-silt Test Section. Waterways Experiment Station, Technical Memorandum No. 3-323, Vicksburg, Miss., March 1951.

5. _____, Investigations of Pressures and Deflections for Flexible Pavements, Report No. 3, Theoretical Stresses Induced by Uniform Circular Loads. Waterways Experiment Station, Technical Memorandum No. 3-323, Vicksburg, Miss., September 1953.

6. _____, Investigations of Pressures and Deflections for Flexible Pavements, Report No. 4, Homogeneous Sand Test Section. Waterways Experiment Station, Technical Memorandum No. 3-323, Vicksburg, Miss., December 1954.

7. _____, Design of Flexible Airfield Pavements for Multiple-Wheel Landing Gear Assemblies, Report No. 1, Test Section with Lean Clay Subgrade. Waterways Experiment Station, Technical Memorandum No. 3-349, Vicksburg, Miss., September 1952.

APPENDIX: EXAMPLE OF THE COMPUTATION OF EQUIVALENT SINGLE-WHEEL LOAD

This appendix provides a detailed example of the method by which theoretical maximum deflections are developed for single- and multiple-wheel assemblies and combined to give a relation between multiple- and equivalent single-wheel loads.

Assume: A dual assembly, 40-in. c-c spacing, 314-sq-in. contact area, A, each wheel.

Then:

$$\text{radius, } r = \sqrt{\frac{A}{\pi}} = \sqrt{\frac{314}{\pi}} = 10\text{-in.}$$

$$\text{spacing in radii} = \frac{40}{10} = 4 \text{ radii between duals.}$$

Fig. 5, which is taken from reference (5), gives deflections for a single load in terms of deflection factor, F, such that:

$$\text{Deflection, } \omega = \frac{prF}{E_m}$$

where

p = load intensity,
E_m = modulus of elasticity.

The following tabulation of deflection factors is taken directly from Fig. 5:

Table 1

Depth in.	Deflection Factors Offset from Center of Single Load		
	Beneath Center	2 Radii or 20 in.	4 Radii or 40 in.
0	1.50	0.39	0.19
r or 10	1.06	0.41	0.20
2r or 20	0.67	0.38	0.20
3r or 30	0.47	0.34	0.20
4r or 40	0.36	0.29	0.20
5r or 50	0.29	0.25	0.19
6r or 60	0.25	0.22	0.17

By the principle of superposition, the deflection beneath one wheel of the dual loading is equal to that beneath the center of a single load plus that at 40 in. (4 radii) offset. Also, deflection beneath the center of the dual assembly is twice that at 20 in. (2 radii) offset beneath the single. Thus, by adding the outer columns of table 1 and doubling the center column, we arrive at the following table of deflection factors:

Table 2

Depth, in.	Deflection Factors	
	Beneath One Wheel of Dual	Beneath Center of Dual
0	1.69	0.78
10	1.26	0.82
20	0.87	0.76
30	0.67	0.68
40	0.56	0.58
50	0.48	0.50
60	0.42	0.44

The maximum deflection beneath one wheel of the dual represents the maximum deflection anywhere beneath the dual loading for shallow depths. Similarly, the maximum deflection midway between the dual wheels represents the maximum deflection anywhere beneath the dual loading for deep depths. The maximum deflection beneath the dual wheels in the transition zone is most easily determined by plotting curves from the data in table 2 on a single plot and visually adding a limiting, or transition, curve, as shown on Fig. 6. This curve could be determined more exactly by superposing deflections beneath the individual wheels of the duals for all offsets between the wheels and selecting the maximum, but the added accuracy does not justify the increased effort.

Table 1 lists deflection factors for points under the center of a single-wheel load. These are the maximum deflection factors for a single load.

The load on a single wheel of the same contact area as one wheel of the dual assembly that produces a maximum deflection equal to that beneath the dual assembly is assumed to be equivalent to the dual loading. We may, therefore, equate deflections from table 1 and Fig. 6. These are expressed as deflection factors such that $\omega = \frac{prF}{E_m}$. By using subscripts s and d to denote single and dual, we may write:

$$\omega_s = \frac{r_s}{E_m} p_s F_s \quad \text{and} \quad \omega_d = \frac{r_d}{E_m} p_d F_d$$

And since ω_s is to equal ω_d, and r_s is to equal r_d (this is true since A_s is to equal A_d),

$$p_s F_s = p_d F_d \quad \text{or} \quad \frac{p_s}{p_d} = \frac{F_d}{F_s}$$

Since contact area is the same for both single and dual, the ratio of total load must be the same as that for unit pressure. Therefore $\frac{P_s}{P_d} = \frac{F_d}{F_s}$. Thus, the ratio of the equivalent single-wheel load to the load on one wheel of the dual assembly is the inverse of the ratio of the maximum deflection factors. In the following table the ratios of dual- and equivalent single-wheel loads are determined for the various depths:

Table 3

Depth in.	Single-wheel Deflection Factor	Dual-wheel Deflection Factor	Load Ratio: Single-to-one Wheel of Dual	Load Ratio: Single-to-dual Assembly
0	1.50	1.69	1.13	0.565
10	1.06	1.27	1.20	0.600
20	0.67	0.89	1.33	0.665
30	0.47	0.70	1.49	0.745
40	0.36	0.58	1.61	0.805
50	0.29	0.50	1.72	0.860
60	0.25	0.44	1.76	0.880

The ratios listed in the right-hand columns of table 3 can be applied directly to the load on the dual assembly (or on one wheel of the assembly) to determine the equivalent single-wheel load for the assembly for the pertinent depth. For example, assume that the dual assembly is loaded with 50 kips and we are concerned with a depth of 20 in.: From table 3 the ratio of single- to dual-assembly loads is 0.665; therefore, the equivalent single-wheel load is 50 x 0.665 = 33.3 kips. Or, we may use the load on one wheel of the dual which is 25 kips. From table 3 the ratio of single load to the load on one wheel of the dual is 1.33. The equivalent single-wheel load is, therefore, 25 x 1.33 = 33.3 kips. The ratios used to relate the 50-kip dual to its equivalent, 33.3-kip, single-wheel load, are valid for all loadings on this dual assembly. Thus, the equivalent single-wheel load for the 20-in. depth for any load can be established.

From the 33.3-kip equivalent single-wheel load and the single-wheel CBR curves, the CBR required at a depth of 20 in. to support the 50-kip dual-wheel load can be determined. For the 100-psi-tire-pressure CBR curves this CBR would be 8.2, and in the same way the CBR values for other loads can be established. By repeating this procedure for various depths, the relation between CBR, thickness of pavement and base, and load can be established and curves drawn for the dual loading selected as an example. This operation can then be repeated for other dual loadings and for other configurations as well.

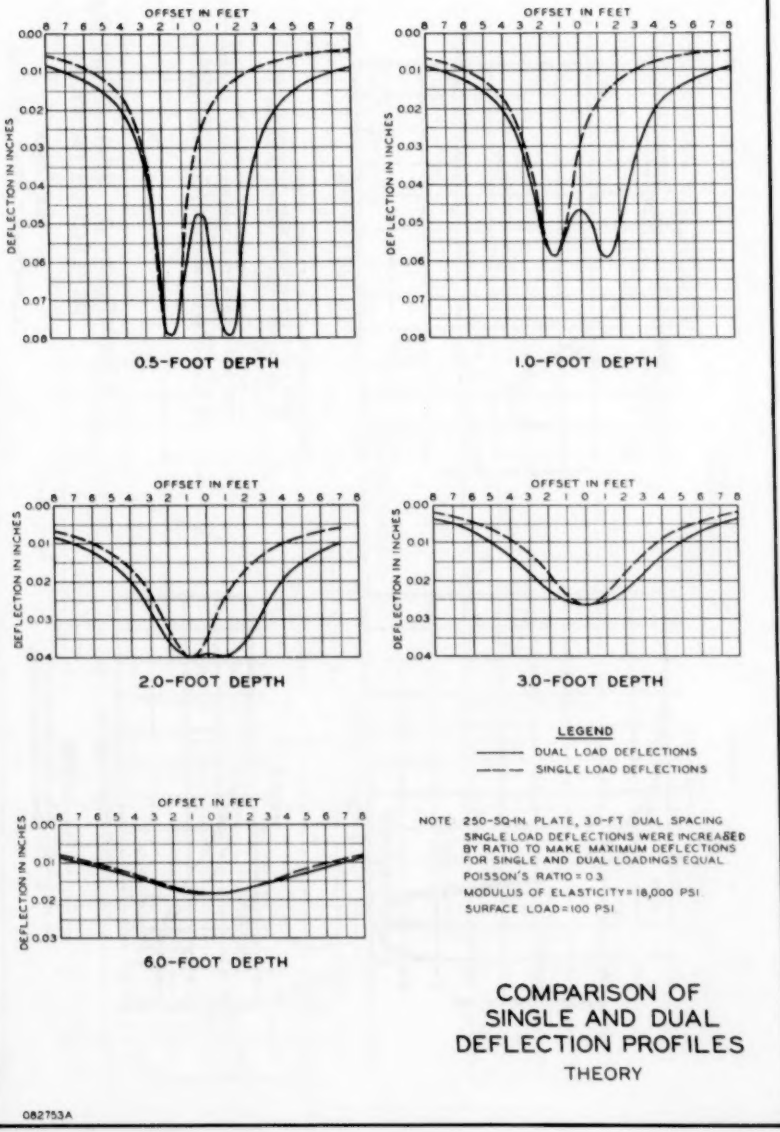
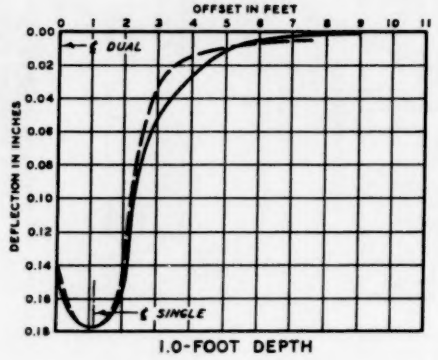
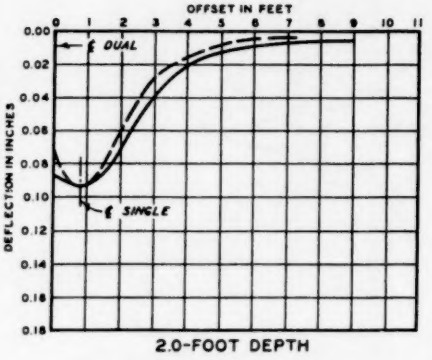

OFFSET IN FEET
8 7 6 5 4 3 2 1 0 1 2 3 4 5 6 7 8
DEFLECTION IN INCHES
0.00
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
0.5-FOOT DEPTH
1.0-FOOT DEPTH
2.0-FOOT DEPTH
3.0-FOOT DEPTH
6.0-FOOT DEPTH
LEGEND
DUAL LOAD DEFLECTIONS
SINGLE LOAD DEFLECTIONS
NOTE: 250-SQ-IN. PLATE, 3.0-FT DUAL SPACING
SINGLE LOAD DEFLECTIONS WERE INCREASED BY RATIO TO MAKE MAXIMUM DEFLECTIONS FOR SINGLE AND DUAL LOADINGS EQUAL
POISSON'S RATIO = 0.3
MODULUS OF ELASTICITY = 18,000 PSI
SURFACE LOAD = 100 PSI
COMPARISON OF SINGLE AND DUAL DEFLECTION PROFILES
THEORY
082753A

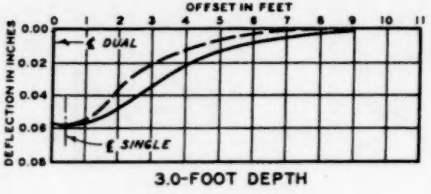
FIGURE 1



1.0-FOOT DEPTH



2.0-FOOT DEPTH



3.0-FOOT DEPTH

NOTE: 500-SQ-IN. PLATE, 3.0-FT DUAL SPACING.
SINGLE LOAD DEFLECTIONS WERE INCREASED BY RATIO TO MAKE MAXIMUM DEFLECTIONS FOR SINGLE AND DUAL LOADINGS EQUAL.
500-SQ-IN. SINGLE LOAD DEFLECTIONS WERE OBTAINED AT HOMOLOGOUS POINTS FROM 1000-SQ-IN. SINGLE LOAD DATA.
DEFLECTIONS WERE AVERAGED FROM THOSE PRODUCED BY 15-, 30-, 45- AND 60-PSI SURFACE LOAD INTENSITIES.

LEGEND

——— DUAL LOAD DEFLECTIONS
– – – SINGLE LOAD DEFLECTIONS

COMPARISON OF SINGLE AND DUAL DEFLECTION PROFILES

TEST DATA

CLAYEY-SILT TEST SECTION

082753C

FIGURE 2

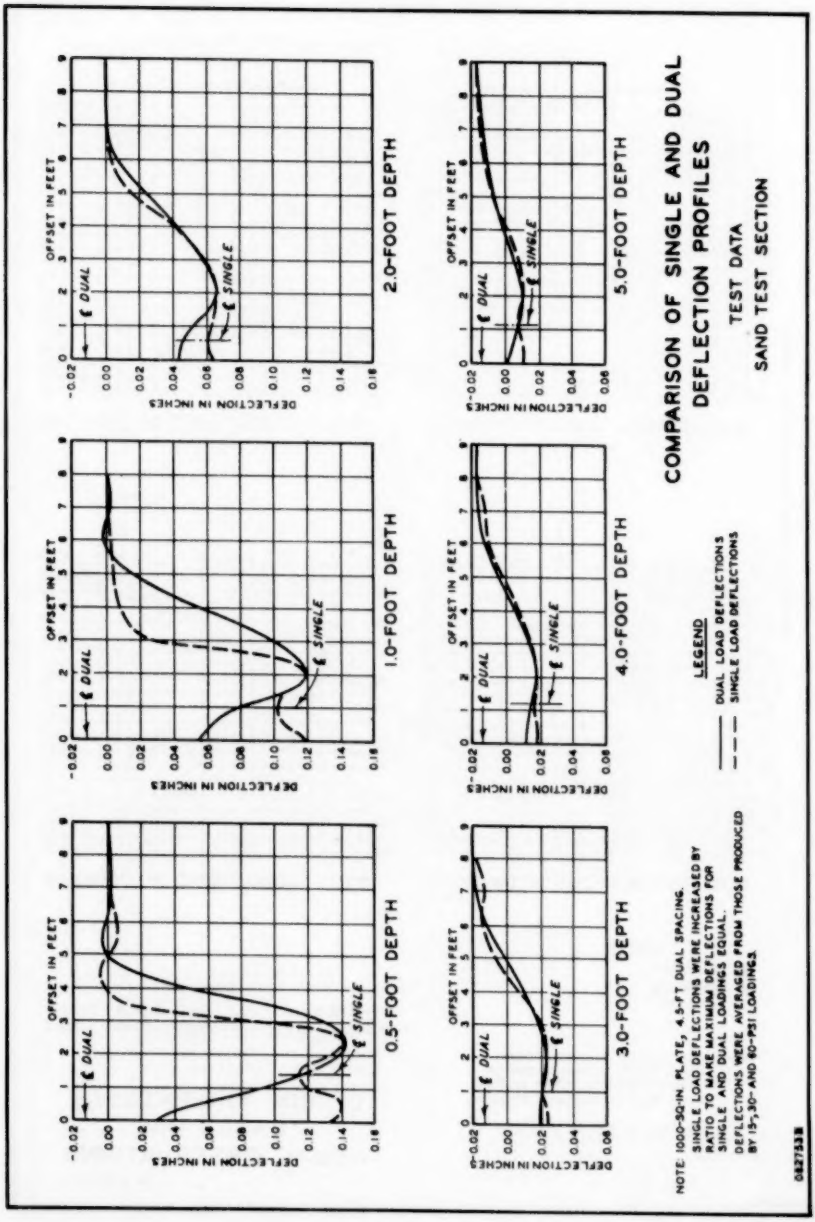

COMPARISON OF SINGLE AND DUAL
DEFLECTION PROFILES
TEST DATA
SAND TEST SECTION
LEGEND
DUAL LOAD DEFLECTIONS
SINGLE LOAD DEFLECTIONS
0.5-FOOT DEPTH
1.0-FOOT DEPTH
2.0-FOOT DEPTH
3.0-FOOT DEPTH
4.0-FOOT DEPTH
5.0-FOOT DEPTH
OFFSET IN FEET
DEFLECTION IN INCHES
℄ DUAL
℄ SINGLE
NOTE: 1000-SQ-IN. PLATE, 4.5-FT DUAL SPACING.
SINGLE LOAD DEFLECTIONS WERE INCREASED BY RATIO TO MAKE MAXIMUM DEFLECTIONS FOR SINGLE AND DUAL LOADINGS EQUAL.
DEFLECTIONS WERE AVERAGED FROM THOSE PRODUCED BY 15-, 30- AND 60-PSI LOADINGS.
0827338

FIGURE 3

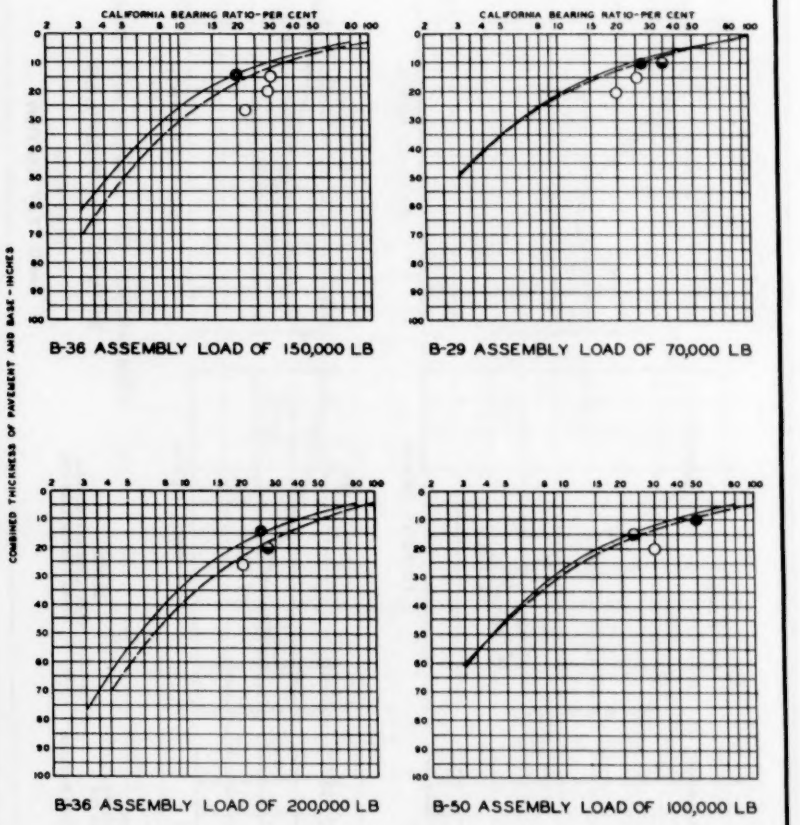
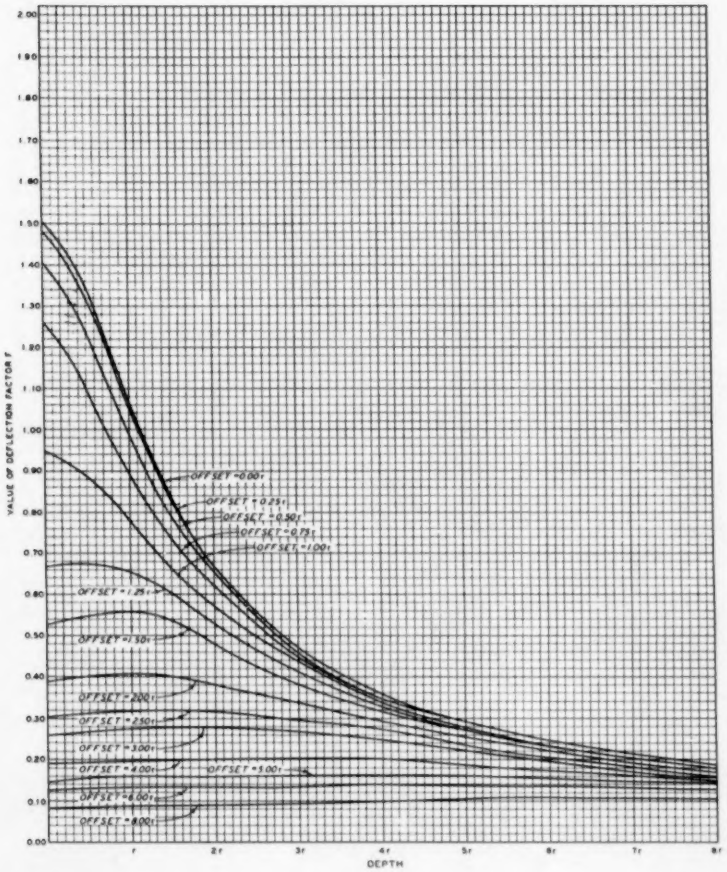

CALIFORNIA BEARING RATIO-PER CENT
2 3 4 5 8 10 15 20 30 40 50 80 100
0 10 20 30 40 50 60 70 80 90 100
COMBINED THICKNESS OF PAVEMENT AND BASE - INCHES
B-36 ASSEMBLY LOAD OF 150,000 LB
CALIFORNIA BEARING RATIO-PER CENT
2 3 4 5 8 10 15 20 30 40 50 80 100
0 10 20 30 40 50 60 70 80 90 100
B-29 ASSEMBLY LOAD OF 70,000 LB
2 3 4 5 8 10 15 20 30 40 50 80 100
0 10 20 30 40 50 60 70 80 90 100
B-36 ASSEMBLY LOAD OF 200,000 LB
2 3 4 5 8 10 15 20 30 40 50 80 100
0 10 20 30 40 50 60 70 80 90 100
B-50 ASSEMBLY LOAD OF 100,000 LB
LEGEND
INADEQUATE
BORDERLINE
ADEQUATE
NOTE: SOLID CURVES INDICATE PRESENT DESIGN CRITERIA; DASHED CURVES INDICATE PROPOSED DESIGN CRITERIA. POINTS INDICATE PAVEMENT BEHAVIOR.
PLATE 17, MULTIPLE WHEEL REPORT NO. 1, MODIFIED TO SHOW PROPOSED DESIGN CURVES
DESIGN THICKNESSES BASED ON VISUAL OBSERVATIONS
090953A

FIGURE 4



$\omega = \frac{p r F}{E_m}$

ω = VERTICAL DEFLECTION IN INCHES
r = RADIUS OF LOADED CIRCULAR AREA IN INCHES
E_m = ELASTIC MODULUS IN PSI
F = DEFLECTION FACTOR
t = DEPTH IN INCHES
p = SURFACE CONTACT PRESSURE IN PSI

NOTE: FOR POINTS BENEATH THE CENTER OF THE CIRCULAR AREA (OFFSET = 0.0r) $F = \frac{3r}{2\left(t^2+r^2\right)^{\frac{3}{2}}}$

OFFSETS MEASURED FROM ORIGIN ALONG X-AXIS

950955 B

DEFLECTION FACTOR F

FOR UNIFORM CIRCULAR LOAD OF RADIUS r
AT POINTS BENEATH THE X-AXIS
POISSONS RATIO = 0.5

FIGURE 5

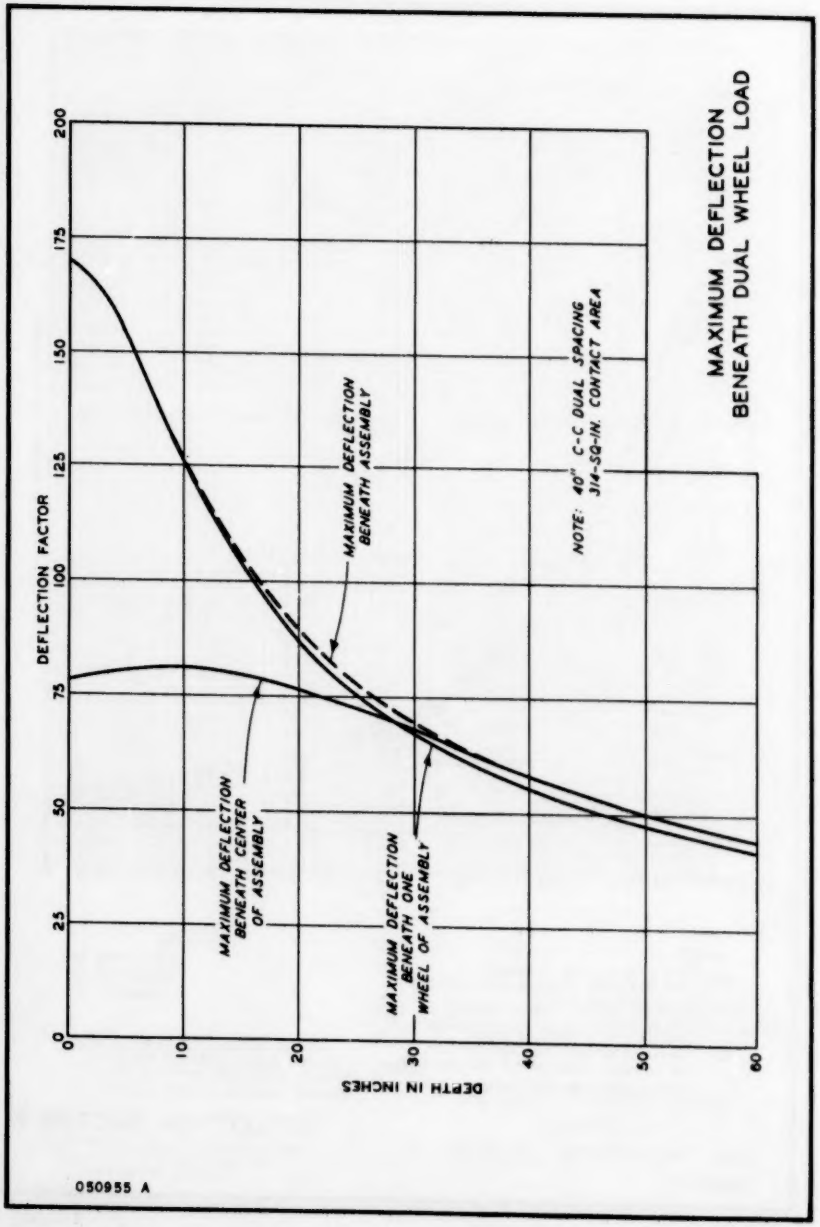

MAXIMUM DEFLECTION
BENEATH DUAL WHEEL LOAD
NOTE: 40" C-C DUAL SPACING
314-SQ-IN. CONTACT AREA
MAXIMUM DEFLECTION
BENEATH ASSEMBLY
MAXIMUM DEFLECTION
BENEATH CENTER
OF ASSEMBLY
MAXIMUM DEFLECTION
BENEATH ONE
WHEEL OF ASSEMBLY
DEFLECTION FACTOR
0
25
50
75
100
125
150
175
200
DEPTH IN INCHES
0
10
20
30
40
50
60
050955 A

FIGURE 6

Paper 1648 May, 1958 SM 2

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

SYMPOSIUM ON "GEOLOGICAL FACTORS IN TUNNEL CONSTRUCTION"
(Proc. Paper 1648)

FOREWORD

In 1952 a Joint Committee of the American Society of Civil Engineers (Soil Mechanics and Foundations Division) and the Geological Society of America was formed to study problems of mutual interest to both societies. Various task committees were assembled for research and study of specific fields of interest. One of these is the task committee on the "Influence of Geological Factors on Tunnel Construction".

The objective of this committee is the "improvement of means for predicting tunneling conditions on the basis of site investigations." In the committee's directive the suggested method of approach is through the assembling of records of tunneling experience, the analysis of these records, and their correlation with geological conditions.

In line with the general function of the committee an initial three papers have been prepared. Two of these papers present details of a more general nature, but within the scope of the committees' activities. The third paper is a specific one involving the use of geophysical (electrical resistivity) methods in preliminary studies of the Lehigh Tunnel, on the new Northeastern Extension of the Pennsylvania Turnpike.

These papers constitute an initial approach to the overall problem and are in no sense to be considered summations of exhaustive endeavors:

Paper No. 1648	"Geological Tools and Techniques" by Arthur B. Cleaves
Paper No. 1649	"Geology and Tunnel Design" by A. B. Reeves
Paper No. 1650	"Geophysical Investigations for the Lehigh Tunnel" by H. L. Scharon and A. B. Cleaves

Note: Discussion open until October 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1648 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 2, May 1958.

SYMPOSIUM ON "GEOLOGICAL FACTORS IN TUNNEL CONSTRUCTION" TOOLS AND TECHNIQUES

Arthur B. Cleaves,[1] M. ASCE
(Proc. Paper 1648)

ABSTRACT

The types of geological investigations essential to delineating potential tunnel problems fall into three principal categories: (A) Library research for all pertinent geological literature and maps of the area; (B) Field reconnaissance and surface and subsurface explorations; and (C) Laboratory studies. New core boring aids such as the bore hole camera and the Trotter-Pajari compass; geochemical techniques, clay technological and petrographic methods are discussed.

INTRODUCTION

The primary purpose of the Task Committee on The Influence of Geological Factors on Tunnel Construction "is to determine if a sufficient knowledge of geological factors before tunnel construction will permit a determination to be made of the necessity for, and the type of supports, and the construction difficulties that might be encountered, such as water, rock bursts, or gas, etc."

The engineer in preparing for any tunnel project is faced with many imponderables concerning the earth materials he must penetrate. His preliminary information may be good, mediocre, or bad.

It is not our purpose in this paper to discuss those qualifications that the engineer must look for in geological or other consulting service. Rather the discussion centers around those geological "tools" or techniques that are, and can be utilized in predicting the soil and/or rock problems that may be encountered.

As will be seen not all of these techniques and methods are applicable in the case of any particular tunnel, but careful selection and implementation of the appropriate ones may permit most of the problems to be anticipated. Obviously considerable leeway is permitted the geological consultant or staff member in the matter of judgment relative to the essential studies that will be made. Because of this personal factor the qualifications of this geologist, insofar as training and experience go, become of paramount importance.

1. Professor of Geology, Washington Univ., St. Louis, Mo.

Types of Investigations

The types of investigations that can be made fall naturally into three principal categories: (A) Library Research concerning all of the available geologic literature and maps relative to the area;(B) Field Reconnaissance and Surface and Subsurface Explorations; and (C) Laboratory Studies.

(A) Library Research: There are relatively few areas in the world today that do not have some written geologic coverage. In the United States this is more complete and in greater detail than in most other countries. Not only are continental and national geologic maps available, but many States are geologically mapped and portions of some in meticulous detail. These maps and their associated reports leave small excuse for lack of general geologic knowledge for almost any area.

Topographic and aerial maps and photographs are available for most of the United States as well as certain other countries. To the geologist trained in photogrammetry (Smith, 1953), (Eardley, 1942) these maps are most revealing in that from them considerable insight into the geologic outcrop pattern, structure, type of bedrock, nature of the soil overburden, presence and absence of faults, and many other important details may be ascertained.

From aerial photographs not only may much of the above data be spotted but the trained observer, by identifying the vegetative plant types, can often determine the gross chemical characteristics of the underlying bedrock, be it igneous or sedimentary. In general lime-loving plants are easily separated from acid soil plants. Many vegetative types have peculiar affinities for particular soil characteristics, hence to a certain degree the plant type serves as an indicator of the kind of underlying bedrock.

(B) Field Reconnaissance and Surface and Subsurface Explorations: (Leroy, 1950), (Hvorslev, 1949). There is no substitute for a preliminary field reconnaissance over the proposed tunnel area. Assuming that the literature source has already been exhausted and some information, at least, has been obtained, the geologist has some clues relative to the types of problems that may be anticipated. These problems fall into very specific groups depending upon whether the tunnel will be a rock tunnel or an earth tunnel (Proctor and White, 1946), (Terzaghi, 1950); and if rock, whether it is in igneous, metamorphic or sedimentary rock. The age of this rock, the topographic landforms that are given expression by it, local drainage characteristics, vegetative cover, and other features give preliminary evidence pertinent to the degree of lithification, gross geologic structure, fault and fracture patterns, and many other details.

The field reconnaissance, on foot where possible, will amplify and crystallize previous data realized from the library research. Forearmed with this information a plan for future surface, subsurface, and laboratory exploration can be formulated.

The on foot traverses, not only over the area of the proposed tunnel line, but also in the tunnel "frame" area, involve among other features use of the rubble (float) line in locating contacts, breadth of outcrop for determining rock structural attitude, and the defining of topographic details not entirely clear from available photographs and maps.

It is obvious that the greater the training and experience of the observer the more valuable the results of the reconnaissance will be in determining the direction and scope of subsequent studies.

Surface Explorations: Depending on the significant details revealed by the

preliminary reconnaissance, a comprehensive surface study may or may not be justified. This can involve detailed geological mapping with many ramifications. Not only may definitive geological mapping of the line above, and in the "frame" area of the tunnel be necessary, but hydrological and geochemical studies may be desirable.

The hydrological studies involve stream drainage and spring locations, pH determinations, and those pertinent features that might be expected to have some effect on the ground waters intercepted by the tunnel bore. The thermal, chemical, and mineralogical characteristics of local spring and/or stream water can contribute very valuable information relative to a proposed tunneling operation. These data can be of utmost importance concerning both design and construction procedures (Happ, 1955).

The geochemical investigations, although not yet a medium of geological exploration that has grown to maturity, can, nevertheless, fill in a gap in a field of study that has hitherto been left largely to guesswork.

The usefulness of geochemistry as a "tool" in determining the type of bedrock below a residual soil depends upon being able to relate some function of the composition of the soil to a given rock type. It is questionable whether this technique has reached sufficient refinement to be useful when the overburden consists of clay-size particles.

Application of geochemical methods requires sampling and analytical techniques suitable to establish a relationship or correlation between the bedrock and the soil. It is based on a logical assumption that a relationship does exist between the composition of the parent bedrock material and the derived, residual soil. The investigator must, at the same time, be fully aware that the factors determining the composition of a residual soil are complex.

Sampling and analytical procedures must be adapted to the requirements of the individual and local conditions. Knowledge of the broad areal geological relationships are generally known and can be used to plan sampling and analysis procedures so as to be efficient and effective.

Circumstances must justify a complete analysis of the soil for the purpose of establishing a definite relationship to a certain rock type. However, it is more likely that a partial analysis may be used for a restricted area or purpose. For example, an unusual concentration of trace elements in a given rock type may be reflected in an unusual concentration of the same elements in the overlying soil. The relative distribution of the specific trace elements in the soil can be determined by sampling and analysis and the location of the rock type from which it was derived fixed accordingly. The accuracy is, of course, limited by local conditions such as slope, soil creep, surface run-off, etc. Attention must be called to the fact that it is essential that all samples be taken from the same horizon in the soil profile so as to minimize the variables caused by weathering.

The total composition, unusual concentrations of elements, and the relative percentage or ratios of constituent elements in residual soils may be used under favorable conditions to determine the nature and extent of the underlying bedrock.

Analytical methods used are thermal, spectrographic, X-ray, and chemical either conventional or the recently developed trace metal method (API, 1951). Although not a geochemical technique the visual or petrographic determination of the heavy minerals present in the soil may be used to trace out geologic formations where rock exposures are rare, and mapping difficult.

Subsurface Explorations: Whereas the following topics are submitted under

"subsurface explorations" they might with equal propriety be classified under surface investigations. Some of these methods, like others previously mentioned, are for the purpose of determining whether bedrock is or is not present. Others are for ascertaining the type and physical characteristics of the bedrock, both with respect to general features, and specific, detailed features.

Geophysical explorations (Heiland, 1940), (Dobrin, 1952) can be applied for determining the position of "top-of-rock", hence earth vs rock excavation, or for more detailed information. In general, either the electrical resistivity, or the seismic methods are used when exploring for the rock-soil boundary. In the East the electrical resistivity method has come into considerable favor for determining the top-of-rock inasmuch as no explosives are required for its application.

These methods can be very helpful in locating and delineating fault and shear zones, igneous bodies, geological structures, and similar phenomena. However, limiting conditions do exist and correlative core-borings should be used where possible and the interpretive results passed upon by an expert.

In earth tunneling the electrical resistivity method, to reasonable depths, can be used to plot the presence or absence of gravel or sandy lenses or beds. The significance of these, with their potentially high water-bearing characteristics is obvious. If such a survey has not been made, but core holes have, and are still open, an electric well log can be made. This gives a much more definitive picture of the sequence of water-bearing and impervious strata or zones, or water bearing fractures and systems in bedrock. Temperature surveys may also be made down these vertical bore holes.

Bore Holes: Perhaps the best known methods of subsurface explorations are bore holes of one type or other (Hvorslev, 1949). Auger borings, varying from about 2 inches in diameter to over 36 inches in diameter, are primarily useful to shallow depths only, and serve a multiplicity of purposes from soil interpretation, to visual inspection (by actual descent into them), and drainage purposes.

Drive sample and diamond drilled core borings are more common and permit more accurate interpretation than most other types of bore holes. In most tunnel investigations, some sort of bore holes will be extremely helpful. In addition to actually seeing samples of the materials to be tunnelled through the accurate determination of the dip and strike of the sedimentary bedrock, or the lineation of the igneous or metamorphic rock can now be made by use of the new U. S. Corps of Engineers "bore-hole-camera". The actual alignment of horizontal and angle bore holes, at depth, can be plotted in both vertical and horizontal planes by the use of the "Tro-Pari bore-hole surveying instrument". (Trotter, 1949).

Other Techniques: It is not always possible or practical to actually investigate the precise line of a proposed tunnel because of its location at great depth below a mountain or range. Nevertheless, in a carefully planned and conducted surface study many clues are available in exposed bedrock, that by interpretation and projection to depth, may prepare the engineer for anticipatory support, and/or water problems.

Careful study and mapping of the joint or fracture system, plotted on what is known as a point diagram (Billings, 1954) may, and usually does, indicate a definite pattern. Depending on the details of this pattern, clues to the gross, and sometimes detailed geologic structures at depth can be determined and predicted. Such data can be definite indicators of the necessity of roof

support, even its type and spacing, and of course, the possible presence or absence of areas of water influx.

(C) Laboratory Studies: A petrographic examination of thin-sections of the exposed bedrock may also be especially revealing in connection with possible subsurface conditions. The physical constituents and weathered status of mineral components in the bedrock, and/or excessively fractured mineral grains may suggest possible wet and weak tunnel sections. They may also indicate potential zones of low pH waters of corrosive characteristics. The textural phenomena observed in the rock thin-sections may have nothing to do with the joint systems, or the regional geological structure.

The applications of clay technology (Pask and Turner, 1955) in relationship to tunnel exploration and construction is another geological "tool" that should not be overlooked. Clays may be present in earth tunnels, or as clay-shales in rock tunnels. In either case they deserve especial study and consideration. Clays are a factor whose role should never be underestimated in tunnel construction. They are notorious for the far-reaching influence that only a small percentage may exert in a soil or stratum. Ten per cent in a plastic soil may render it somewhat plastic, and as much as 50 per cent will impart essentially all of the clay properties that are significant in engineering.

The determination of moisture content and plasticity in soils is now common practice. Less common is the search for the underlying causes of soil conditions that may be hazardous. Effective remedial measures may often depend upon a follow through to determine the soil component at fault—and the clays are open to suspicion.

When problems of squeezing or of swelling ground arise in a tunnel excavation, such factors as moisture content and rebound upon release of confining pressure, etc. come to mind. But, back of these are the natural properties of clay minerals reacting to new environmental conditions occasioned by the excavation.

The moisture content of soils is in part regulated by the percentage of contained clay. The affinity of clays for water, retention of water, plasticity, and the capacity for swelling, are all closely related features.

The relatively strong surface charges on the surfaces of individual clay platelets demand satisfaction, and water, usually present to some degree, and being mobile, is a natural respondent. Its presence, in a sheath several molecules thick and varying in its retention from practically rigid attachment near the clay particle to loosely bound layers further away, acts as a lubricant, and one which is reluctantly given up.

Certain positively charged ions also find themselves incorporated, sandwich-like, between the clay platelets. These control to a considerable extent the degree of hydration of the clay. They usually bring along an envelope of water of their own, and in the case of one of the montmorillonite clays, can attract available water to the extent of expansion of the individual mineral particle many times its original size. This expension, or swelling, is a forceful one and is reversible. Montmorillonites occur as particles one micron in diameter, consequently offer a tremendous surface area when present en masse or even dispersed in the pore spaces of silts and sands. The wetting of deposits containing those clays could be disastrous to a tunnel or shaft-sinking operation.

Fore-knowledge of the presence of minerals of this type and their location and abundance would allow for planning the circumvention of the problems they forecast in tunnel excavation.

The occurrence of the montmorillonite, kaolinite, and illite clay minerals is widespread. Their properties vary and their detection and identity often present difficulties. However, through the means of the petrographic and electron microscopes, x-ray diffraction and differential thermal equipment, and studies on base exchange and dehydration, it has become possible to overcome most of these difficulties.

CONCLUSION

With the various geological (Schultz and Cleaves, 1955), soil mechanics (Terzaghi and Peck, 1948), and engineering "tools" and techniques available for tunnel exploration there would appear to be little excuse for the failure of such costly ventures today. Nevertheless, failures there are, but it is believed that many of these derive from the unpredictable human element and the lack of understanding of the methods of exploration available for the asking.

BIBLIOGRAPHY

1. A.P.I., 1951, Reference clay minerals. Preliminary Reports Research Project 49, API, Columbia University, N. Y. (Available API, 50 West 50th Street, New York).
2. Billings, Marland P., Structural Geology, pp. 108-115, Prentice-Hall, N. Y., 1954.
3. Dobrin, M. B., 1952, Introduction to geophysical prospecting, 435 p., McGraw-Hill.
4. Eardley, A. J., 1942, Aerial photographs, their use and interpretation, 203 p., Harper & Bros., N. Y.
5. Happ, S. C., 1955, Engineering Geology Reference List, Geol. Soc. Amer., Bull., vol. 66, pp. 993-1030.
6. Heiland, C. A., 1940, Geophysical Exploration, 1013 p., Prentice-Hall, N. Y.
7. Hvorslev, M. J., 1949, Subsurface exploration and sampling of soils for Civil Engineering purposes, Waterways Experiment Station, Vicksburg, Mississippi, (available through Engineering Foundation, 29 W. 39th St., New York 18, N. Y.)
8. Leroy, L. W., 1950, Subsurface Geologic Methods, p. 1158, Colorado School of Mines, Golden, Colo.
9. Pask, J. A. and Turner, M. D., 1955, Clay and Clay Technology, 326 p., California, Div. Mines (San Francisco), Bull. 169.
10. Proctor, R. V. and White, T. L., 1946, Rock Tunneling with Steel Supports, 271 p., Commercial Shearing and Stamping Co., Youngstown, Ohio.
11. Schultz, J. R. and Cleaves, A. B., 1955, Geology in Engineering, 592 p. John Wiley & Sons, N. Y.
12. Smith, H. T. U., 1943, Aerial photographs and their applications, 372 p. D. Appleton-Century Co., N. Y.

13. Terzaghi, Karl, 1950, Geologic aspects of soft ground tunneling, p. 193-209, Applied Sedimentation, John Wiley & Sons, N. Y.

14. Terzaghi, Karl and Peck, R. B., 1948, Soil Mechanics in Engineering Practice, 566 p., John Wiley & Sons, N. Y.

15. Trotter, Charles, 1949, Description of Tro-Pari borehole surveying instrument, p. 1922, A.I.M.E., Trans. Vol. 181.

Paper 1649 May, 1958 SM 2

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

SYMPOSIUM ON "GEOLOGICAL FACTORS IN TUNNEL CONSTRUCTION"
GEOLOGY AND TUNNEL DESIGN

A. B. Reeves,[1] M. ASCE
(Proc. Paper 1649)

ABSTRACT

The geologist-engineer relationship in tunnel design and construction is outlined. Methods of investigation, selection of tunnel location and factors to be considered in the preparation of designs and specifications for tunnels are discussed and illustrated by examples.

INTRODUCTION

Geology has been defined as "the science which treats of the structure of the earth's crust and the physical forces operating in connection with changes which have taken place in the past or may be still occurring".

During the past few decades, the science of geology has assumed increasing importance in planning, design, and construction of certain types of engineering projects. This development has led to the creation of a branch of the profession frequently termed "engineering geology". The engineering geologist has the responsibility of providing the basic information on the composition, structure, and stability of the natural material on which an engineering structure may be supported, or through which it may be required to be constructed.

For the past two years the author has spoken before the geology class in the graduate school of the Colorado School of Mines at Golden, Colorado, on engineering problems relating to the conveyance of water. During these talks a question frequently asked might be stated:

"What does the tunnel engineer expect the geologist to know?"

This is difficult to answer in a few words, but by attempting to answer the

Note: Discussion open until October 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1649 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 2, May 1958.

1. Chief, Irrigation Operations Div., Bureau of Reclamation, Denver, Colo.

question the importance of geology to tunnel design may be emphasized.

The engineer usually asks the geologist to obtain the necessary information for design and construction and to supply data on the probable structural soundness in a given foundation material. To obtain the information, the geologist must be familiar with the tools available to him and the methods of using these tools. He must also be familiar with design and construction techniques and practices required for various types of materials. In addition, he must have a good understanding of the costs of construction.

Geologist - Engineer Relationship

The geologist - engineer relationship in tunnel design and construction is closer than in most any other field of engineering because the problems of the tunnel engineer are largely associated with the rock, soil, and ground water, which comprise the geologist's special field of professional activity. The engineer requires geological information for the most effective and economical planning; the geologist is challenged to supply and analyze this information. The more precise and quantitative this information is, the more useful it will be to the engineer in the planning, design, construction, and operation of engineering works.

The engineer has the responsibility for the decision as to the kind of structure and how it is to be built. The geologist has the responsibility for interpreting the geologic conditions to which the structure must be adapted. The geologist's interpretations accomplish their greatest usefulness when he understands the engineering viewpoint and objective, the alternatives of design, and the procedures of construction that can be made available for use. The geologist must also be aware of the technical facilities and limitations of engineering procedure.

Examination and Investigation

Procedures for the geological examination of foundations for dams, bridges, buildings, or other types of large structures are fairly well established. Foundation materials for these structures are investigated by drilling, test pitting, and testing of cores or samples, by seismographic methods, and scientific examinations of surface exposures. Interpretations must be based on training, experience, and judgment.

Rarely does the excavation for the foundation of such structures extend to great depth. Ordinarily, drilling at dam sites will extend to a depth of one-half of the anticipated height of the structure, this depth being modified by the conditions encountered. Comparable criteria are established for foundation exploration for other structures.

For diversion tunnels at dam sites or vehicular tunnels under rivers or for grade separations, geologic examinations usually can be made by core drilling or by test pits or borings. However, coring and sampling are not ordinarily practicable for tunnels which extend through ridges. Such geologic exploration is impracticable, if not impossible, for tunnels to be driven through mountain ranges.

The Bureau of Reclamation has constructed many tunnels in the western part of our country for the conveyance of water for irrigation and for power purposes. Outstanding tunnels constructed on Bureau of Reclamation projects include the Alva B. Adams Tunnel on the Colorado-Big Thompson Project in Colorado, 13.1 miles long; the Tecolote Tunnel on the Cachuma Project in

California, 6.4 miles long; the 6.2-mile long Duchesne Tunnel on the Provo River Project in Utah; and the 6-mile long Gunnison Tunnel on the Uncompahgre Project in Colorado. These tunnels—which, incidentally, are the four longest on Bureau projects—as well as others built by the Bureau, presented difficult geological problems.

The Gunnison bore, completed in 1909, after 4 years of intensive work, was driven through granite, quartzite, gneiss, and shale, and layers of sandstone, coal, and limestone, first by manual labor, then later with power equipment. The Duchesne construction—a 3-year undertaking, completed in 1952—imposed unusual difficulty because of massive quartzite that was about five times harder than the average rock and because of the phenomenon of "popping" or exploding of the quartzite when it was stress relieved by drill holes.

The Tecolote Tunnel was holed through in January 1955 after 5 years of unusually difficult excavation work. Inflows of water, sand, methane gas, and high temperatures and humidity, all combined to tax the ingenuity of the geologists, engineers, and construction forces.

Tunnels constructed by the Bureau range from 6 to 56 feet in diameter. They include the 4 diversion tunnels at Hoover Dam, which are each 4,000 feet long and 56 feet in diameter. Geological exploration for all but a few of the tunnels on Bureau projects was necessarily limited to short reaches near the portals where the cover was shallow enough to permit drilling at reasonable depths. With few exceptions, the cost of drilling to a depth greater than 350 feet has been prohibitive.

Seismographic investigations determine the depth of alluvial cover and provide other useful information. However, they fall far short of drill cores in determining the type of rock or other material to be encountered at depth. On the Alva B. Adams Tunnel the Bureau used seismographic studies supplemented by drilling to determine if glacial overburden deposits exposed on the surface would be encountered near the proposed tunnel portal.

Locations

Generally, the grade, elevation, and alinement, coupled with comparative cost estimates of alternative routes, dictate where the bore is to be made. Sometimes a slight shifting of the tunnel alinement will provide a better portal location or permit a better angle of intersection with a known fault. We should not expect the geologist to locate exactly a weak zone or its extent at a depth of even a few hundred feet. What should be asked for is his opinion that such a zone may or may not be anticipated.

When the location of a tunnel has been established and the engineering plan laid out, explorations at the portals should be made. Drilling along the alinement to reasonable depths will indicate conditions in portal areas. Beyond these limits, the physical characteristics of the surface must be studied and mapped to indicate the dip and strike of the rock formations and joints or faults revealed by outcrops. Projection to tunnel levels and interpretations of the relationship of surface to depth conditions can be made. Geophysical determination of the characteristics of rock formations exposed along a tunnel line, supplemented with other geologic data, have been used in projecting surface geology to tunnel level.

Preparation of Designs and Specifications

In the preparation of designs and specifications for tunnel construction, it

is essential that the designer have as much information as possible concerning the various types of material and formations that may be encountered. For this reason, geological explorations should be coordinated with the design studies as the work progresses. Frequently, design considerations will influence the extent of the necessary geological explorations. Similarly, economies can be realized when a knowledge of the design factors involved may eliminate unnecessary exploration. It is difficult to recall a tunnel site that has been overexplored. Money spent on thorough investigations is saved many times over in economical and efficient design and construction.

The presence of ground water in tunnelwork is of major concern to design and construction engineers. Geologic determination of the location of water and of the volume of the expected flow is a prime requisite in tunnel exploration. If water is to be encountered in certain types of rock structures, the practicability of grouting may be an important consideration. If flows cannot be eliminated in this manner, drains and pumps may be required. Groundwater problems are recognized to be in the field of the geologist, who by training and experience will be prepared to give professional assistance to the engineer.

Frequently the tapping of ground water may have implications beyond the engineering phases of the tunnel project. Instances of interference with domestic water supply are not uncommon. One case has been the subject of a damage claim where the tunnel was nearly 3 miles from the alleged affected area. Although this may be more of a legal than an engineering consideration, it is definitely a problem for the engineering geologist.

The existence of faults in tunnels is not unusual, nor does it necessarily result in construction difficulties. Fault displacements may range from a few inches to miles. Displacements may be very narrow and sharply defined or may involve zones of hundreds of feet in width. Careful geological examination will usually reveal the presence of displacement. The geologist can often forecast a condition with sufficient accuracy for adequate structural design. He can also assist in the preparation of plans to cope with construction difficulties that may be anticipated. Faults that contain crushed rock zones or clay seams or gouge may indicate the presence of water. In a zone of fractured or heterogeneous material, the existence of water may decrease the stability of the material. We should not expect the geologist to anticipate accurately all actual conditions. However, he should be prepared to give further assistance on examination of the heading during exploration when closer inspection is possible.

If a fault is known to be active, the engineer and geologist should seriously consider some other type of construction. Surface structures damaged by an earth movement will be more adaptable to maintenance or repair than any type of underground construction. Most active faults have been identified, and the geologist should warn the engineer of their existence and of the hazards to subsurface structures in the vicinity. Unless there are extraordinary reasons to the contrary, any underground construction in the vicinity of an active fault should be avoided. For example, the lack of adequate geologic investigations resulted in the location of some major railroad tunnels in active faults; subsequent earthquakes caused such severe dislocations to the tunnels that the road had to be relocated.

At best, the measurement of loads or pressures in a tunnel is difficult, and unless the rock is uniform in structure, these pressures will vary to a great degree under apparently similar conditions. It is the responsibility of

the designer to design the tunnel lining or the structural supports to resist the greatest load to be anticipated. He should enlist all means at his disposal to identify the reaches of the tunnel requiring support.

Occasionally, tunnels are subjected to hydrostatic pressure. The computations of these pressures caused by static loads and added effects due to surge may be accurately determined. Steel reinforcement to carry these combined loads can be designed with the usual design factors. Frequently, however, in tunnels driven through well consolidated rock, appreciable savings in reinforcement can be made if the depth of cover and the quality of the rock are sufficient to resist the hydrostatic pressures.

In two tunnels constructed by the Bureau of Reclamation in recent years, detailed examination of the walls and the roofs and stress measurements by experienced engineers and geologists led to the decision to eliminate reinforcing steel. A combined savings of nearly one million dollars was thus realized.

Construction

Most large construction projects are now built by contractors under the system of competitive bidding. Advance knowledge of all types of tunnel support anticipated or required may have a favorable reaction on bid prices. To provide a basis for competitive bidding, it is necessary that design and specifications contain all practicable information on conditions to be met and on the types of structures proposed to resist the anticipated load. It is usually necessary that the inside dimensions of a tunnel section to be fixed, at least within certain limits. As most tunnels are lined with concrete, any variation of the support will require a change in the excavated section.

The success of modern tunnel construction can be attributed in large part to the resourcefulness and skills of the contracting industry. Contractors with the assistance of experienced tunnel engineers and geologists have developed efficient tunnel excavation equipment, procedures, and methods for difficult and hazardous tunnel construction. The drilling jumbo, mechanical mucker, improved drill bits, millisecond delay blasting caps, transportation facilities, and ventilation techniques have all been steadily perfected. Steel supporting ribs have practically eliminated the use of timber for structural uses, except for lagging or blocking. As a result of these improvements, tunnel construction is now virtually routine.

The standardization of equipment and construction methods has greatly influenced design practices. However, when fractured zones or faults are encountered, the use of standardized equipment and methods may not be adequate. Other unusual conditions, such as large inflows of water, may require special equipment and techniques. To provide tolerable working conditions in the Tecolote Tunnel, for example, major modifications in unwatering facilities and of ventilating and dehumidifying equipment were required.

Improvements in construction and geologic techniques have immeasurably influenced safety in tunnel work. During construction of the Gunnison Tunnel more than 40 years ago, several men lost their lives and many more sustained serious injuries. The Duchesne Tunnel, about the same length as the Gunnison Tunnel, was completed a few years ago with only a few minor injuries to the workmen.

The geologist should be familiar with the latest construction techniques and practices. The preparation of detailed logs of actual tunnel construction is one of the best methods to obtain this knowledge. The detailed log is

important in tunnel construction as it shows the geologic structures and rocks encountered and describes the support required. This record has lasting usefulness in the maintenance and repairs that may be necessary in future years.

Economic Considerations

The economic coordination of geologic studies and their interpretation and application to engineering must be sound from a practical and scientific approach. Alterations of alinement, section, or other properties are practicable only if they achieve the desired objective without loss of engineering feasibility or economic balance. An alteration of alinement was made in one tunnel so that a greater proportion of the tunnel would lie in shale and so that the crown would be in a hard, flat-lying sandstone bed.

It is frequently possible to show a theoretical advantage of unsymmetrical cross sections to conform with the structure of the rock anticipated or actually encountered. Modifications of alinement and shape made in mining operations would be objectionable in water conveyance tunnels due to controlling factors related to the function of the structure. In unlined tunnels, overbreak or irregular shape may be seriously detrimental to hydraulic efficiency and may reduce the capacity to such an extent that a greater diameter is required with resulting increase in cost. If the tunnel is to be lined, and the size and shape have been designed for a certain capacity requirement, any unnecessary overbreak represents needless removal of rock to be replaced by expensive concrete or other material.

CONCLUSION

This paper began with the question "What does a tunnel engineer expect the geologist to know?" It may well conclude with a statement "What the tunnel engineer should know!" Because of the influence of geological considerations on the design and costs of tunnel sections, supports, and appurtenant features, it is important that engineers engaged in tunnel work be familiar with elemental geology. With a limited amount of geological knowledge, the design engineer may take advantage of the known geology of the region through which a proposed tunnel is to be located. From this understanding, he will be enabled to decide what further detailed explorations or geological interpretations may be advisable. He will also be better qualified to understand a geological report and to translate this understanding to his engineering considerations.

It is not the intention of this paper to imply that the engineer should engage in the field of the geologist or vice versa. However, by a better appreciation of the professional qualifications of each other, more intelligent results can be obtained, and greater efficiency in the work and economy in the final accomplishment can be realized.

Paper 1650 May, 1958 SM 2

Journal of the

SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

SYMPOSIUM ON "GEOLOGICAL FACTORS IN TUNNEL CONSTRUCTION" GEOPHYSICAL INVESTIGATIONS FOR THE LEHIGH TUNNEL

H. LeRoy Scharon* and Arthur B. Cleaves,* M. ASCE
(Proc. Paper 1650)

ABSTRACT

Geophysical (electrical resistivity) investigations were conducted in the portal areas of the Lehigh Tunnel on the new Northeastern Extension of the Pennsylvania Turnpike System. These studies were made for the purpose of indicating the approximate depths to solid bedrock. The results showed (a) bedrock much deeper than originally postulated, (b) the most desirable areas for the rock portals where bedrock approached the ground surface.

INTRODUCTION

The Lehigh Tunnel site, one of the major structures of the Northeastern Extension of the Pennsylvania Turnpike System (Fig. 1), is located in Lehigh and Carbon Counties southeast of the town of Lehighton and just before crossing the Lehigh River. The length of the tunnel from portal to portal is approximately 5,600 feet; however, geophysical investigations were confined to the portal areas only, which together total a distance of 2,200 feet of linear investigation.

In the first investigation of the tunnel site some question arose as to the depth to and condition of bedrock in the portal areas. To quickly determine these factors, geophysical investigations involving the electrical resistivity method were used for a rapid reconnaissance and to guide drilling which would ultimately have to be carried out in order to locate desirable rock portal faces.

At each portal site electrical resistivity station centers were established with eleven (11) in the south portal area and seven (7) in the north portal area.

Note: Discussion open until October 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1650 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 2, May 1958.

* Professors of Geology, Washington Univ., St. Louis, Mo.

These stations were not always on center line but were in the immediate vicinity. Electrical resistivity changes were observed with depth.

Topography

Considerable topographic relief is evident along the center line of the tunnel site. This relief, approximately 800 feet, has its maximum value at the county line. Of particular interest are the changes in slope along the center line. On the south portal side a major change in slope occurs at station 1345+00 and in this case represents the start of a talus slope overlying both residual soil and bedrock. On the north portal side a change in slope occurs at approximately station 20+00, whereas another may be observed at station 30+00. The first change in slope is probably associated with a rock type change but the latter is a break in slope due to accumulated, transported material. These talus or transported materials fall in the critical area of rock portal faces.

Geology

The bedrock in the line of the proposed tunnel consists of shales, sandstones, quartzites and conglomerates. These strata occur in the form of thick beds, and in interfingered, cross-bedded and transitional relationships. The strata range (south to north), from Upper Ordovician, Martinsburg shales, through the Ordovician, Bald Eagle "conglomerate", the Silurian, Tuscarora quartzite and quartzitic sandstones, the Silurian, Clinton shales, sandstones, siltstones, and quartzites and the Silurian, Bloomsburg shales. All of these strata are of marine, sedimentary origin and in general are firm, strongly indurated rock.

The geologic structure (Fig. 2) is simple, and shows constantly and relatively uniformly dipping beds averaging from about 30° to 40° inclination from the horizontal toward the south. In the south portal area this dip is readily observed by the conglomerate at station 1347+00 which has a measured dip of 45° to the south. In the north portal area, a tracer bed in the Bloomsburg Formation was cut in two drill holes at stations 29+32 and 30+42. Occasional units show southerly dip as high as 55° and as low as 20°. The bedrock strikes roughly east-west.

These relatively uniformly dipping strata are all overturned and comprise the south limit of a synclinal structure, the axis of which lies north of the tunnel area.

The mountain slopes above the proposed tunnel line are littered with stone rubble characteristic of much of the harder, underlying bedrock. Because of the sloping ground and the effect of gravity and creep phenomena, rubble representing the harder strata under the higher slopes has moved downhill and masks the shales and "softer" rock in the portal areas. Such materials on the portal sections are noted as transported.

South Portal Area

From about 1343+76 to about station 1346+40 on the floor of the tunnel, the bedrock consists of the Martinsburg shale of Ordovician age. This rock generally is a firm, solid, gray to black, siliceous shale and siltstone. The bore hole at station 1346+50 encountered the Martinsburg shale. Between elevations 761 and 756 feet the bedrock is a poorly consolidated, iron-stained,

sandstone-conglomerate. The bedrock underlying this zone is a very hard, conglomeratic quartzite, the Tuscarora formation.

From about stations 1346+40 to 1347+40 all of the core boring data indicate broken and crushed rock.

From a geophysical point of view, the overburden situation as revealed in the thirteen (13) core borings present a rather complicated interpretation problem. As noted on Fig. 2, there is a variable thickness of transported material covering a likewise variable thickness of residual soil. These further lie over a varying thickness of a weathered zone of the Martinsburg shale before sound rock is encountered. The geologic contacts are difficult to interpret as will be shown later.

North Portal Area

In this area seven (7) core borings were made after the completion of the geophysical work, as was the case in the south portal area. From about station 30+98 to about station 29+05 on the floor of the tunnel, the bedrock indicated consists of the Bloomsburg formation of Silurian age. This rock is chiefly firm limey shales, sandy shales, fine-grained sandstones and siltstones. The Clinton-Bloomsburg contact is established at the southernmost bed of limey shale.

As in the south portal area overburden conditions are complicated enough to present a serious problem of detailed interpretation of the electrical data. Again, transported material rests on top of residual soil and it in turn rests on a thin layer of weathered rock.

Geophysics

The geophysical method of approach to the problem was the utilization of the electrical resistivity technique. Essentially this consists of passing an electrical current into the ground through two current electrodes and then measuring the potential of the current between two potential electrodes. If the spacing between the various electrodes is the same and all electrodes are in a straight line, then the apparent resistivity of the rock involved in the measurement is computed by the formula,(1)

$$\rho = 2\pi AE/I$$

where ρ is the resistivity in ohm centimeters, E is the potential in millivolts, I is the current in milliamperes and "A" is the electrode spacing. The depth of measurement, through experience, has been found to be approximately equal to the value of "A".

The Gish-Rooney type electrical meter and the Wenner configuration were used in these investigations. Actually the technique is not a new one and in recent years has seen considerable application in engineering problems.(2) The Pennsylvania Turnpike Commission has used the technique on its Philadelphia, Western, Delaware and the Northeastern Extensions of its turnpike system.

Field Procedure

Because depth to and conditions of bedrock were factors to be determined by the application of the electrical method, the spread between the electrodes

was increased at 1-1/2 foot increments to a distance of 12 feet and then at 3-foot increments to the total distance measured. In this manner a variation in apparent resistivities was determined at each station center and with these variations an attempt was made to interpret them geologically.

The electrode spreads with two exceptions were run at right angles to the direction of the center line of the tunnel site. This was done in order to keep the effect of topography at a minimum. However in the two cases (south portal area) the apparent resistivity profiles were essentially the same.

All of the electrical measurements were carried out before actual drilling. This is a normal procedure; however, in order to give complete interpretation some key drilling is needed for correlations and interpretations.

Results

All apparent resistivities were computed and plotted as a function of "A". These data when computed and plotted were further correlated with each other primarily to note general changes in resistivity profiles with others in the same area. Such changes will give a general indication of subsurface conditions without making specific interpretations as to actual depths to geologic contact.

When drilling was completed these same data were again investigated and specific resistivity breaks correlated with geologic changes.

Interpretations of resistivity results in the south portal area (Fig. 2) were extremely difficult, especially for stations 8, 9, 10 and 11 because of the sharp contact between formations and the thinning of the various conditions above sound rock. On the north portal area interpretations were less difficult although only stations 2, 3 and 5 are presented.

South Portal Area

Resistivity profiles 12, 13 and 14 are shown correlated with drilling results on Fig. 3. These correlations, however, were not made until drilling was completed.

It is pointed out, however, by disregarding actual correlations and observing each profile with respect to the others that certain general resistivity changes are apparent. Profile 12 has a sharp high at a relatively shallow depth and then a sharp decrease in apparent resistivity. Profile 13 has a high resistivity at a shallow depth, broader and then a sudden decrease in resistivity around 40 feet. Profile 14 has a broad resistivity high and again a sudden decrease around 40 feet. These changes, on the basis of experience, are directly associated with a thickening of the overburden from station 12 to 14.

It was suggested, before geophysical work was done, that bedrock was of the order of 10 feet in depth in this particular area. After geophysical measurements and before drilling it was interpreted that bedrock in the area of stations 13 and 14 was of the order of 40 feet in depth; however, at station 12 much shallower. After drilling, bedrock was found to be deeper than 40 feet at each station. In the original interpretation the depth suggested to bedrock (shale) was not to sound rock but to the top of the weathered shale. Thus it can be seen that the electrical resistivity method is unable to delineate between weathered shale and sound shale since both have a very low resistivity. This would not be the case if bedrock had been a limestone or sandstone.

After drilling, an attempt was made to correlate the top of the weathered shale and the top of sound rock. These correlations in general agree; however,

there are other resistivity breaks on each profile which could readily be picked as the contacts.

North Portal Area

Resistivity profiles 2, 3 and 5 (Fig. 4) are shown correlated with the drilling results.

In the north portal area it was suggested before geophysics was started that depth to bedrock was extremely shallow, less than 10 feet. However, after completion of electrical measurements it was stated that bedrock was shallow only in the vicinity of stations 1 and 2 and that stations 5, 6 and 7, all run to depths of 60 feet or more, did not reach bedrock. The correlation sheet showing stations 2, 3 and 5 bear out this interpretation. Station 2 has a sharp high resistivity at a shallow depth and then drops off rapidly at a depth of 15 feet. Station 3 has a sharp resistivity high at a shallow depth but drops at a slower rate to a depth of 25 feet and then very rapidly below this. On the other hand, station 5 does not reach a sharp high resistivity peak nor does it decrease with depth as in the case of 2 and 3. In the interval of total measurement at this station the values of the resistivity are too high to suggest a shallow shale bedrock; thus it was interpreted that bedrock somewhere between 4 and 5 dropped to a depth greater than 60 feet.

After drilling was completed, correlations were made as shown (Fig. 4). However, in the case of station 3 it was difficult to pick the break for bedrock since the break for residual soil occurs at 24 feet while the top of residual soil was actually 25 feet from drilling data. This particular profile (Fig. 5) was empirically treated according to the Moore[3] method. From the cumulative resistivity curve a definite break is noted at a depth of 31.2 feet. This correlates with a depth of 31.8 feet to shale as determined from the drill data. The residual soil break is not apparent on the cumulative resistivity profile.

CONCLUSIONS

The resistivity application on the Lehigh Tunnel site did indicate that bedrock was much deeper than postulated in the portal areas. These data also indicated that bedrock was a shale since very low apparent resistivities were measured. If bedrock had been a sandstone or limestone, the apparent resistivities would have been much higher.

After drilling, an attempt was made to correlate the electrical resistivity results. It was found that preliminary interpretations on the south portal area were made at the top of the weathered shale rather than on sound rock. Such a condition is not indicated on the resistivity profiles.

Finally, measurements of this nature can be made and preliminary interpretations made to give a generalized picture of the subsurface conditions. These measurements may also be used to determine drill sites.

REFERENCES

1. Wenner, Frank, "Method of Measuring Earth Resistivity", Bureau of Standards, Scientific Paper 258 (1915).

2. "Symposium on Surface and Subsurface Reconnaissance," American Society for Testing Materials, Special Technical Publication No. 122 (1951).

3. Moore, R. Woodward, "An Empirical Method of Interpretation of Earth-Resistivity Measurements," A.I.M.E., T.P. No. 1743 (1944).

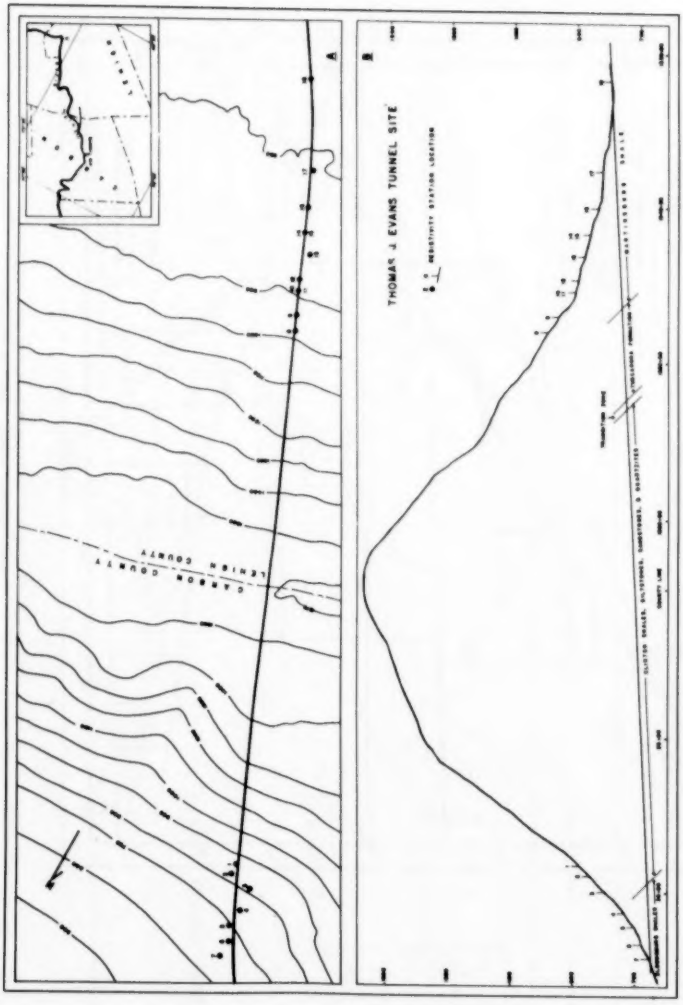


Figure 1. Plan and Profile Views of the Lehigh Tunnel showing locations of geophysical points of observation.

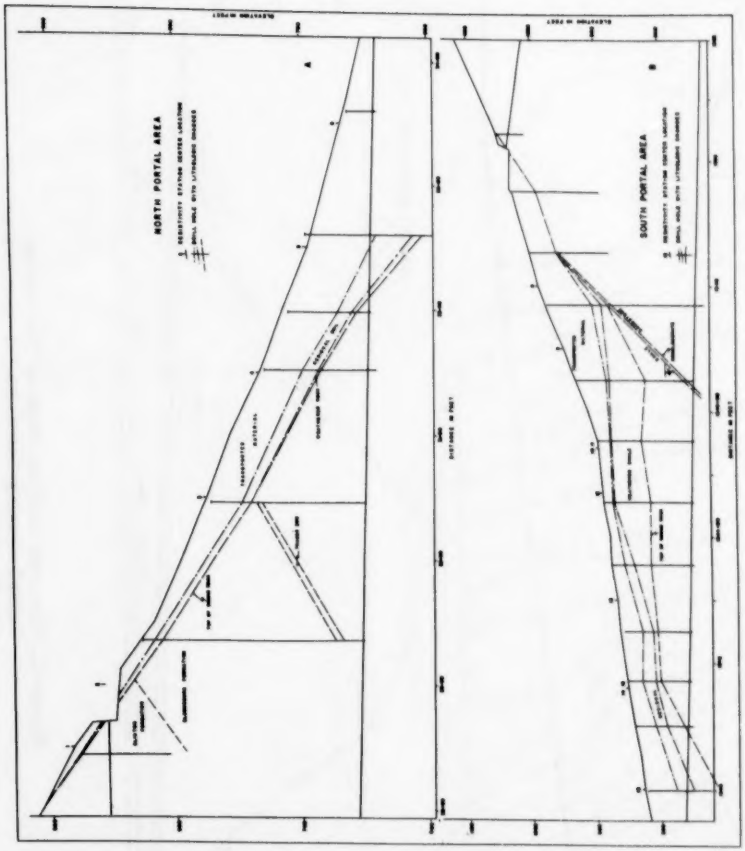


Figure 2. Geological Sections in the North and South Portal Areas, Lehigh Tunnel.

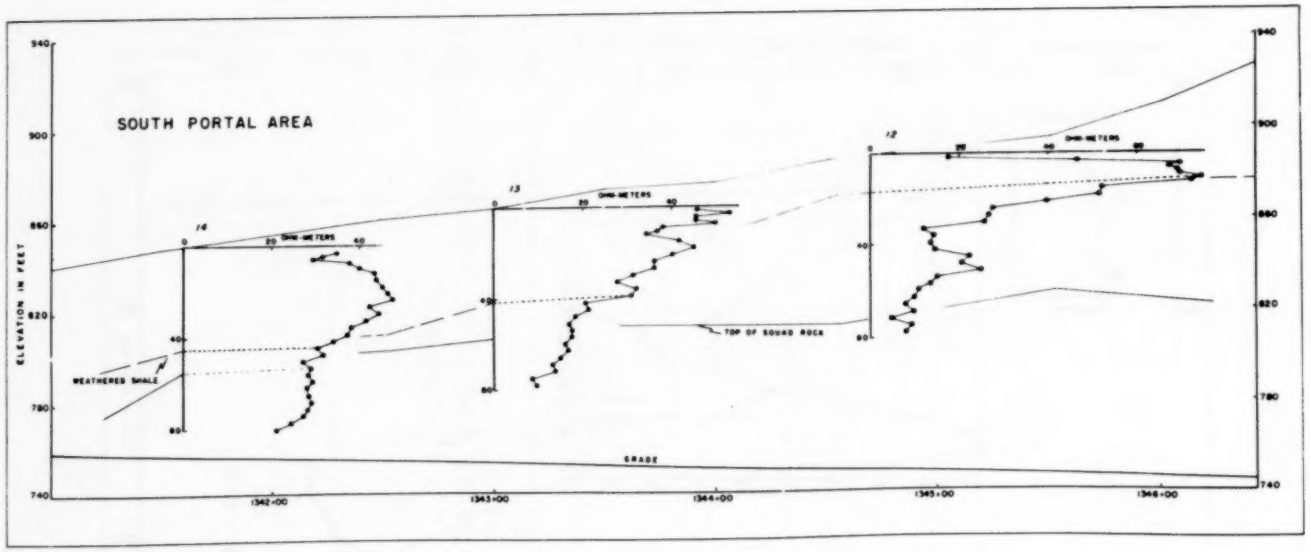


Figure 3. Correlation of Geophysical Data and Drilling Data in the South Portal Area, Lehigh Tunnel.

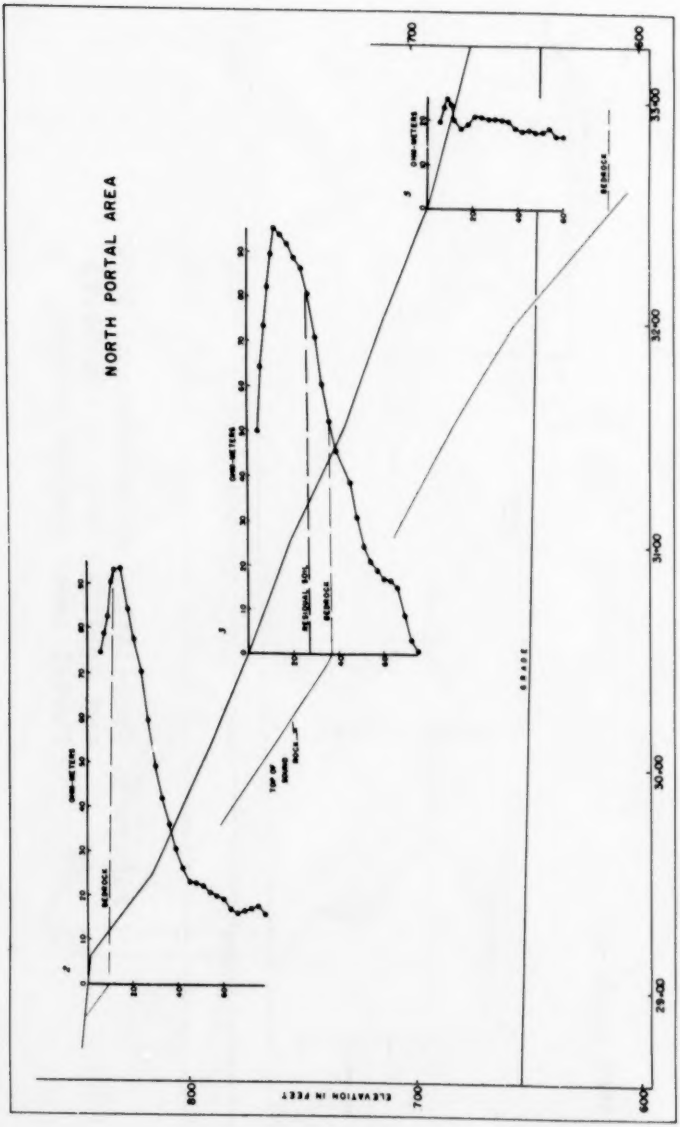


Figure 4. Correlation of Geophysical and Drilling Data in North Portal Area, Lehigh Tunnel.

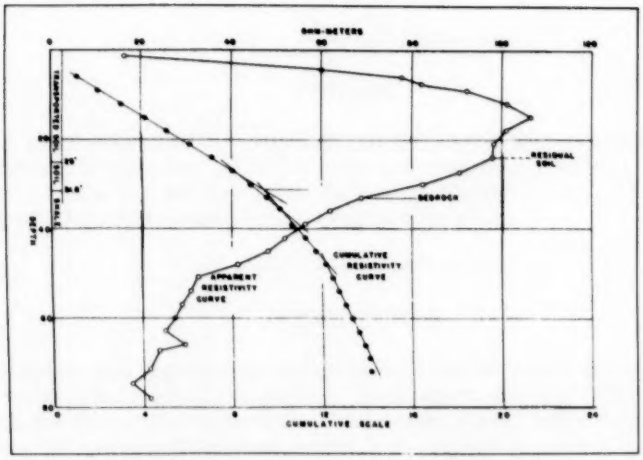


Figure 5. Example of Interpretation of Resistivity Data.

Paper 1654 May, 1958 SM 2

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

THE STRUCTURE OF COMPACTED CLAY[a]

T. William Lambe,[1] M. ASCE
(Proc. Paper 1654)

SYNOPSIS

This paper describes the nature of clay structure, a term referring to the arrangement of particles and the electrical forces acting between them. The effects of forces and environmental factors on structure are inferred from the principles of colloid chemistry and crystal chemistry.

INTRODUCTION

The nature and engineering significance of the structure of compacted clay are treated in a series of two papers. This paper, the first in the series, describes the nature of clay structure; the second paper, which follows this one, employs the concepts presented herein in an attempt to explain the fundamentals of the engineering behavior of clay.

"Structure" means the arrangement of soil particles and the electrical forces acting between adjacent particles. The structure of a clay determines its properties. The engineer must, therefore, consider structure in order to understand the fundamentals of soil behavior. He needs an understanding of structure especially if he tries to predict the effects on soil properties of time, pressure and changes in environmental conditions. The increasing study of soil structure by engineers in many of the countries of the world indicates a growing recognition of the importance of structure.

The term "structure" was initially limited to the arrangement of soil particles. The inclusion of electrical forces as a component of structure, as done in this paper, seems necessary. Progress on elucidating the force component of structure has been made only in recent years, primarily because the

Note: Discussion open until October 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1654 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 2, May, 1958.

a. Presented at a meeting of the ASCE, Chicago, Ill., February, 1958.

1. Director, Soil Stabilization Lab., and Assoc. Prof. of Soil Mechanics, Massachusetts Inst. of Technology, Cambridge, Mass.

underlying sciences—colloid chemistry and crystal chemistry—are relatively young. The importance of the arrangement of soil particles, however, was recognized many years ago (Terzaghi (1925), Casagrande (1932), and Hvorslev (1938)). Hvorslev (1938) postulated the effect of one-dimensional compression on the alignment of clay platelets. Measurements described in this paper show Hvorslev's postulations were correct.

Studies on compacted clay can be of immediate practical value as well as of value to the basic understanding of the behavior of fine-grained soils. Compacted clays, both with and without added stabilizers, are being used more and more in engineering works. Plastic clays are, for example, being placed in earth dams and highway subgrades by soil engineers more often than in the past, sometimes by choice and sometimes by necessity. As the use of and severity of imposed loads on compacted clay increase, the need for a better understanding of the properties of the clay becomes greater.

This series of papers should throw much light on the important question, "Should a fine grained soil be compacted wet or dry of optimum moisture content?" Some of the effects of molding water on behavior of a soil will be demonstrated. The best placement moisture for a given soil depends on the properties desired in the particular problem at hand.

As will be brought out in this paper there are fundamental similarities between the structures of compacted and natural clays. Studies on compacted clays can, therefore, improve our knowledge of naturally sedimented clays. There are experimental advantages to using compacted instead of natural clays for studying the fundamental properties of clays.

While the following material deals primarily with clays, most of the concepts hold also for soils containing silt and/or clay. Colloidal phenomena are, of course, less important in soils of low or no plasticity. The paper is limited to inorganic soil components.

As the reader will see, several facets of structure are not fully understood. The author does not, therefore, present this paper as the final word, but rather as a stimulant and, hopefully, a guide to further studies on this complex subject.

I. Aggregation and Dispersion of Soil Particles

A. Electrical Forces

Many groups of scientists, especially colloid chemists and crystal chemists, have given much thought to the electrical forces acting between atoms (see, for example, Evans (1948), Pauling (1940), Rice and Teller (1949), and Kruyt(1952)). The author in an earlier paper (1953) gave a short description of some of these forces. They fall into three categories: primary valence bonds, hydrogen bonds and secondary valence forces.

Primary valence bonds, which are the strongest, hold atoms together in the basic mineral units. The three most important bonds to the soil mineralogist are the ionic (an exchange of electrons by the linked atoms), the covalent (sharing of electrons by the linked atoms) and the heterpolar (in effect, part ionic and part covalent, since it results from an unequal sharing of electrons by the linked atoms). Some crystal chemists do not recognize the heterpolar bonds as a separate type but merely a mixture of ionic and covalent bonds. The bonds linking silicon and oxygen atoms in the silicate minerals are heterpolar, i.e., 50% ionic and 50% covalent. Since the ionic and heterpolar bonds bring about atomic groups which are not electrically symmetrical,

they act like magnets. This electrical nature of the soil minerals is most important, as we shall see.

The hydrogen bond occurs when an atom of hydrogen is rather strongly attracted by two other atoms.* The best example of the hydrogen bond is the linkage between water molecules. Attracted to the oxygens in neighboring water molecules, each hydrogen atom links its water molecules to others.

The hydrogen bond is considerably weaker than primary valence bonds. It operates not only in water, but also in other areas of interest to the soil engineer. It is the main linkage between the basic units of certain minerals, the most important of which is kaolinite.

The primary valence and hydrogen bonds are both too strong to be broken by the stresses an engineer would normally apply to a soil system. This is not true, however, of the secondary valence forces, which are much weaker than the other two. Another difference between the secondary valence forces and the other two types is the fact that secondary valence forces can act over relatively large distances.

The secondary valence forces (also called van der Waals forces) between units of matter arise from electrical moments existing within the units. They are quite similar to those acting between two short bar magnets. In certain relative positions the magnets repel each other and in others, they attract. Since there are more attractive positions than there are repulsive positions, the net effect is attraction. In a similar fashion the net effect of secondary valence forces is attraction.

Because of the nonsymmetrical distribution of electrons in the silicate crystals (arising from the heterpolar bonds), these crystals act as a large number of dipoles. They can attract other dipoles, the most important being water. The nonsymmetrical configuration of the water molecule makes it dipolar.

The water molecule is a permanent dipole since its polar nature arises from the position of the atoms in the molecule. Other kinds of dipoles are induced and statistical. When a normally symmetrical molecule is placed in an electric field it becomes polar. The electric field can cause a nonpolar unit to become an induced dipole.

Since the electrons in an atom are not stationary but vibrating, they periodically assume unsymmetrical positions. Symmetrical molecules periodically become polar, i.e., statistical dipoles.

These secondary valence forces are of vital concern to the soil engineer since they contribute to clay strength and cause soils to hold water. Much of the remaining portion of this paper is concerned with these important electrical forces.

Another type of primary valence linkage which must be considered is the electrostatic attraction or repulsion of electrically charged units of matter. The negatively charged soil particle attracts cations; cations and anions attract each other; cations repel each other; and anions repel each other. Those electrostatic forces are similar to primary valence bonds in that they involve units with net electrical charges. They can, however, act over much larger distances than can ionic bonds.

In summary, the main importance to the soil engineer of primary valence and hydrogen bonds is that they are the seat of electrical forces. Of much

* Hydrogen bonds are only important when oxygen, nitrogen or fluorine atoms are present.

more concern to the engineer are secondary valence and electrostatic forces since they are greatly influenced by applied stresses and by changes in the nature of the soil-water system.

B. Colloid Stability

Electrical forces of the types just described act between all adjacent soil particles; they become important only when they are large in comparison to the forces related to mass. Since the electrical forces act only near the particle surface and since many of them result from discontinuities at or near the surface, they are significant when surface area per mass, i.e., specific surface, is high. When a particle has a specific surface large enough to cause the electrical forces to dominate the mass forces, it is termed a "colloid". The approximate size range of colloids is 1 micron to 1 millimicron (10A). Clay-size soil particles are thus usually in the colloidal range.

Clay particles carry a net negative charge (primarily caused by isomorphous substitution*) which is balanced by exchangeable cations. When the clay is dry, the cations cluster at the clay surfaces to neutralize the particles. When a clay particle is placed in water the cations (plus a smaller number of anions) swarm around the colloid as illustrated in Fig. 1. Fig. 1b shows the distribution of ions with distance from the clay particle surface and Fig. 1c is a curve of electrical potential** with distance. The swarm of counterions is called the double layer and the colloid plus double layer is termed the micelle. The counterions are also called "exchangeable" since they can be replaced.

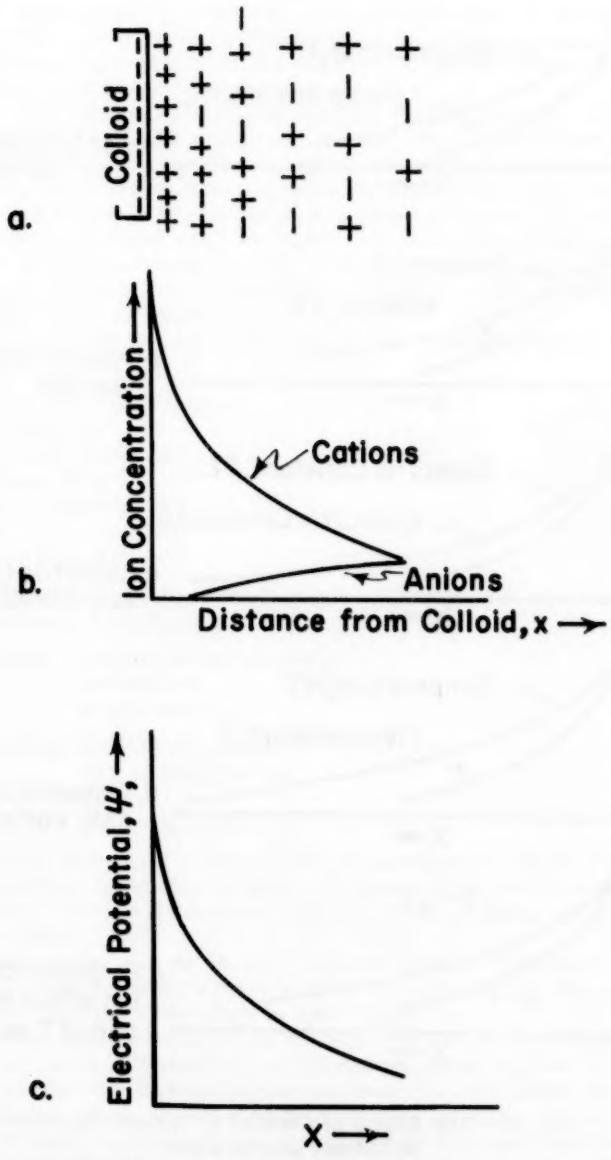
The theoretical expression for electrical potential in the double layer was developed independently by Gouy (1910) and Chapman (1913) and is often called the Gouy-Chapman theory. This theory shows how the potential-distance curve, i.e., double layer, varies with characteristics of the dispersion medium. These effects are illustrated in Fig. 2.***

---

* Isomorphous substitution is the proxying of one atom for another in a crystal without a change in crystal form. For example, the difference between the mineral pyrophyllite and the mineral montmorillonite is that statistically every sixth aluminum in pyrophyllite is a magnesium in montmorillonite. Pyrophyllite is electrically neutral; the proxying of magnesium with a valence of +2 for aluminum with +3 gives montmorillonite a net negative charge of -1 for every substitution.
Isomorphous substitution should not imply that one mineral is formed in nature from another by simply removing one atom and replacing with another. The differences in minerals with the same crystal forms helps the study of minerals; they do not necessarily indicate a generic process.

** The electrical potential at any point is the work necessary to bring a unit charge from an infinite distance to that point. Moving a charge from low to high potential requires a work input equal to a force times the distance. In other words, the force required to move the charge is the slope of the potential-distance curve.

*** Figs. 2a through 2d show the effect on the double layer of varying only the characteristic noted while holding the other characteristics constant. The dielectric constant of water cannot be held constant, however, while varying temperature. Fig. 2e shows the direct effect plus the indirect effect of varying the temperature.



DIFFUSE DOUBLE LAYER

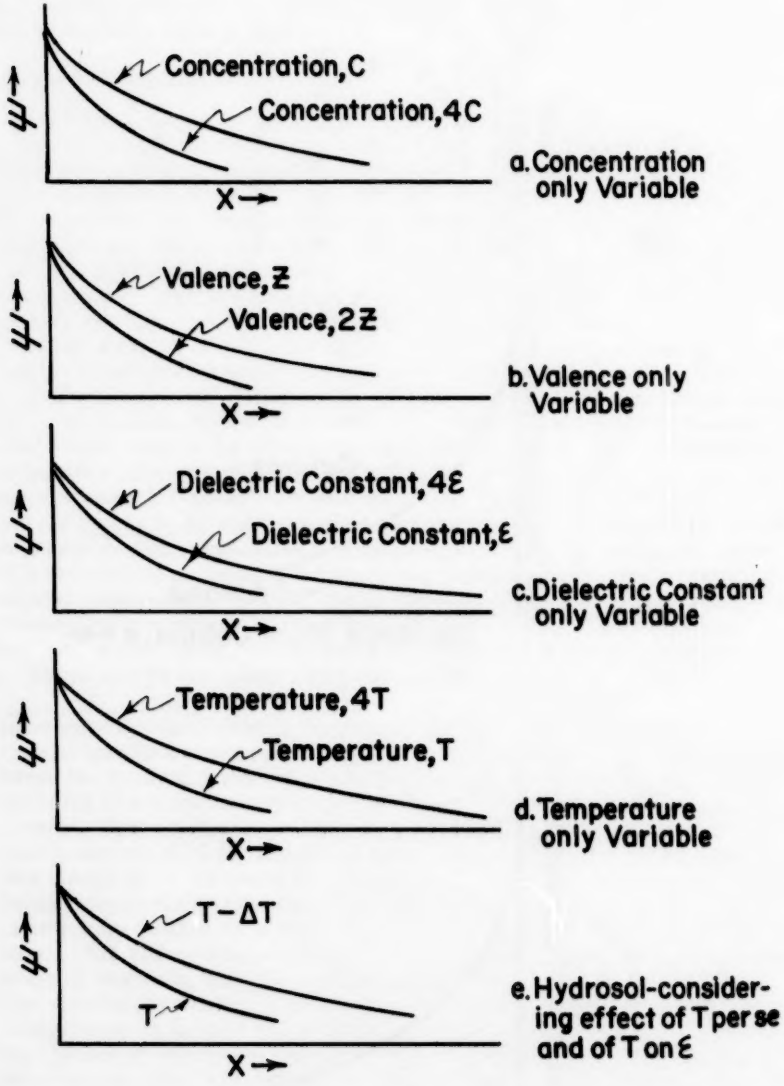


Fig. 2 — The Effects of Changes in System Properties on Diffuse Double Layer

Since the clay particles carry net negative charges, they electrostatically repel each other. This repulsion between two adjacent particles becomes effective when the particles approach each other close enough for the double layers to overlap. Using the Gouy-Chapman theory, colloid chemists have developed theoretical expressions for the electrical repulsion between colloids as a function of the distance between them.

Secondary valence attractive forces also act between particles. Combining the repulsive and attractive potential expressions, chemists have developed equations relating the total potential energy to interparticle spacing for both spherical and plate colloids. These equations (Kruyt, 1952), Bolt, 1955) show that if the potential energy is reduced when adjacent particles approach each other, they will so approach and "flocculate", i.e., form aggregates. If the energy of the system increases when the particles approach each other, they will move apart, or disperse. Changes in the variables of the colloidal system can cause aggregation or dispersion.

Among the variables in a soil-water system which affect colloidal stability are:

1. Electrolyte Concentration
2. Ion Valence
3. Dielectric Constant
4. Temperature
5. Size of Hydrated Ion
6. pH
7. Anion Adsorption

The Gouy-Chapman theory tells us, in general terms, that a tendency toward flocculation is usually caused by:

Increasing:	electrolyte concentration ion valence temperature
Decreasing:	dielectric constant size of hydrated ion pH anion adsorption.

The influence of the first four variables follows from the Gouy-Chapman theory of the diffuse double layer, since a decrease in the double layer thickness reduces the electrical repulsion, which, in turn, causes a tendency toward flocculation.

The smaller an ion plus its "shell" of hydration water, the closer it can approach the colloidal surface; thus the smaller the hydrated ion, the smaller the double layer and the more likely is flocculation.

The pH of the pore fluid affects the net negative charge on a soil particle by altering the extent of dissociation of OH groups on the edges of the particle. High pH encourages the dissociation and increases the net charge, thus expanding the double layer; low pH does the reverse. Lowering the pH, therefore, tends to cause flocculation and raising it tends to cause dispersion.

The adsorption of anions, especially polyvalent anions, increases the charge on the particles and thereby tends to cause dispersion.

Most of the effects of varying the characteristics of the dispersion medium on the stability of soil colloids can be demonstrated with a soil-water

suspension in a test tube. Changing successively the items listed above can affect aggregation or dispersion* in accordance with the predictions based on the theory. Fig. 3 illustrates these effects.

C. Non-Salt Flocculation

The Gouy-Chapman theory assumes that the colloid charge is spread uniformly over the entire colloid surface. This implies that the edges of the soil platelets carry the same charge as the flat surfaces or that the ratio of edge area to plate area is small. (This ratio is not always small; it can be as large at 1/5.) If, as in the Gouy-Chapman theory, the edge effects are ignored, it can be shown that the position of adjacent flocculated clay plates which gives the minimum free energy is parallelism. Evidence, however, that the edges of clay particles can be oppositely charged from the faces follows:

1. Thiessen (1942) has shown with electron micrographs that negative gold micells were adsorbed only at the edges of kaolinite particles. He also showed that positive silver colloids were adsorbed on both the flat surfaces and the edges. This proves that the edges are positively charged and the faces negatively charged.
2. Michaels (1957) has shown that kaolinite adsorbs negative ionic groups (polyphosphates) with a resulting increase in ion exchange capacity from 2 m.e./100 g to 5 m.e./100 g.
3. Soil colloids have a lower electrophoretic mobility than expected (van Olphen, 1950), which indicates interference with the double layer by edge charges.
4. Schofield and Samson (1953, 1954) have shown that deflocculation of kaolinite in water occurs at the point of zero Cl^- adsorption. At low pH values, where flocculation occurs, there is strong positive Cl^- adsorption; at high pH, dispersion negative Cl^- adsorption take place. These facts are evidence of edge-to-face flocculation.

There are also indications that flocculated sediments are not composed of plates in a parallel array:

1. A flocculated sediment is very loose, i.e., has a low density; this means that the colloids cannot be packed in a parallel arrangement since this efficient packing would give a high density.
2. Fahn, Weiss, and Hofmann (1935) proved that montmorillonite particles in a thixotropic gel were in contact. They removed the water from the gel by quick freezing followed by sublimation. The removal of water by this means left a rigid structure of essentially the same volume as the gel. Such a porous structure could hardly be obtained from parallel particles.

The inescapable conclusions are therefore: 1) the edges of clay particles can carry a charge opposite to that of the faces, and 2) flocculated

* In general, the larger the sediment volume (lower the unit weight) after sedimentation, the higher the degree of flocculation. Some of the largest sediment volumes can decrease markedly with time or disturbance since the flocs are relatively weak and are partially destroyed by sediment overburden or externally applied forces.
The effects of ion valency, ion size and pH are complicated by ion exchange reactions.

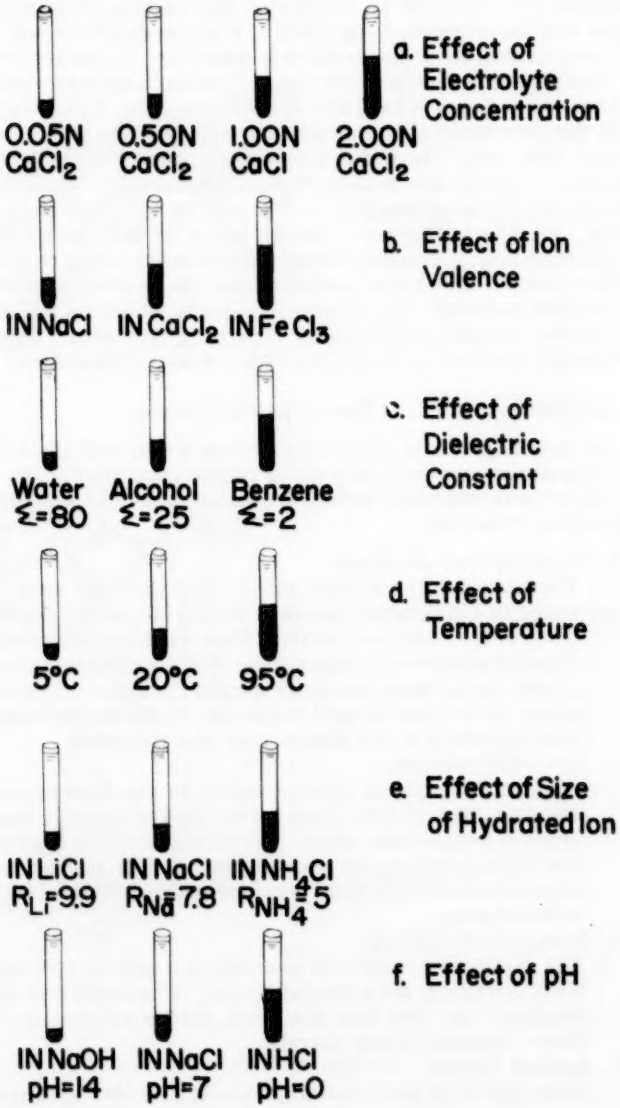


Fig. 3 — Effects of System Characteristics on Soil Sediment

sediments are not composed entirely of parallel particles. Two types of flocculation, salt type with an orientation approaching parallelism, and a non-salt type with an orientation approaching a perpendicular array, can exist. These orientations and a dispersed orientation are illustrated in Fig. 4.

The presence of both types of flocculation in the same colloidal system can be demonstrated with kaolinite in water. As Fig. 5 illustrates, pure kaolinite in distilled water flocculates by the edge-to-face mechanism to give a very loose sediment. The addition of a small amount of electrolyte results in dispersion; further salt results in salt-flocculation. Adding salt to the system increases the concentration of anions which gather at the edges of the particles. Because of this anion concentration the positive charge of the particle is effective over a smaller distance and thus its ends are less able to cause flocculation. The same neutralization can be obtained with small negatively charged particles, e.g., montmorillonoids (Schofield and Samson, 1954). The use of the principle of streaming bire-fringence* shows a higher degree of particle parallelism with salt than with non-salt flocculation.

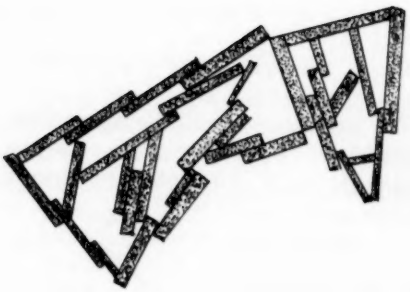
D. Limitations of Colloid Theory in Soil Masses

The soil engineer is almost always concerned with large masses of natural soil rather than with very dilute suspensions of colloids. There are, therefore, several important factors neglected in colloidal theories which he must consider,** namely:

1. Particle Size and Shape
   The particles in a natural soil are not all of the same size and shape as the theories assume. Nearly all soils contain particles of many shapes and many sizes. Most soils contain more than one mineral and several impurities. Silt-size particles occur in most plastic soils; these non-plate shaped particles affect the arrangements of the plate shaped particles. Colloidal phenomena are much less important in silt than in clay size particles.
2. Soil Concentration
   The concentration of mineral matter in soil fluid is much higher than assumed; in fact, most natural inorganic soils contain far more mineral matter than water. There is considerable physical and electrical interference among neighboring particles. The theory of colloid interaction should consider many particles, not simply two adjacent ones.
3. Interparticle Spacing
   The average interparticle spacings in a natural soil are the lower limit of validity for colloidal theory. When particles are almost touching, i.e., less than 10A apart, forces not considered by the Gouy-Chapman theory operate.
4. Applied Forces
   Most clay-size particles are, indeed, colloidal when they are in

* Shaking a suspension of particles with a high degree of parallelism will cause a streaked or nonhomogeneous appearance. This appearance is due to the reflection of light from a large number of particles at the same orientation.

** These factors are in addition to the assumptions of the characteristics of individual colloids made in the theories, such as: uniform surface charge, ions are point charges, etc.



a. Salt Flocculation

b. Non-Salt Flocculation

c. Dispersion

Fig. 4 — Sediment Structures

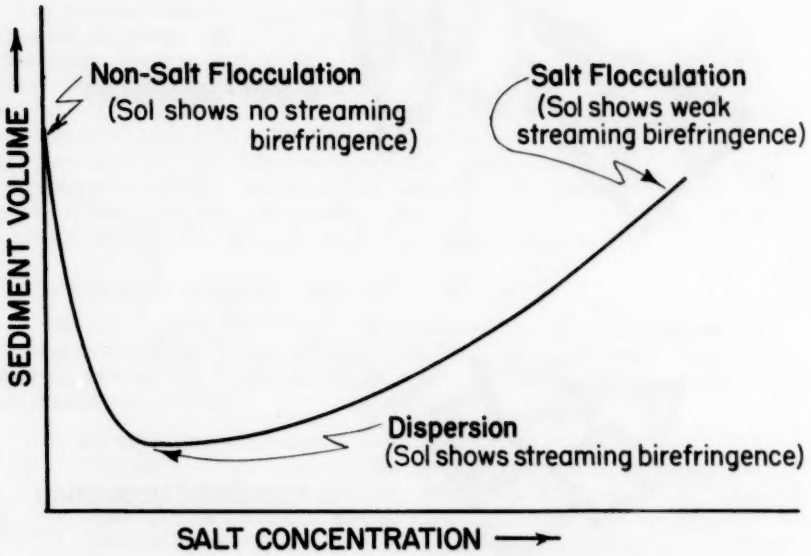


Fig. 5 — Flocculation and Dispersion of Kaolinite in Water

dilute suspensions; in large clay masses, however, there are very important forces which are external in origin. While the mass of an isolated clay particle results in a force that is inconsequential relative to the colloidal forces acting, the accumulated effect of overburden in a natural deposit is not inconsequential. The various other intergranular forces which the soil engineer encounters, such as those which occur from overlaying structures or from moving water, can be of great importance in determining the behavior of clay. The force system acting between soil particles is influenced not only by the forces arising from the nature of particles but also by those arising from non-compositional sources. Colloid theories usually consider only the former.

Because of these four factors, plus other less important ones, the various attempts to use numerical equations based on colloidal theory in soil engineering problems have met with failure (except in a few cases where carefully selected materials and environments were employed). The colloidal principles are, however, extremely useful to the soil engineer since they are vital to an understanding of the fundamental behavior of fine grained soils.

II. Arrangement of Soil Particles

The preceding section considered primarily one component of structure, namely, the electrical forces between partices; this section treats the second component, the arrangement of soil particles.

A. Shapes of Mineral Crystals

Using the electron microscope, clay mineralogists have made excellent progress in determining the sizes and shapes of the clay mineral crystals. Shapes of some of the common clay minerals are:*

Kaolinite	- sheets (often regular hexagonal sheets)
Halloysite	- tubes of curled sheets
Montmorillonite	- flaky sheets
Nontronite	- laths
Illite	- irregular flaky sheets
Attapulgite	- narrow fibers
Chlorite	- sheets
Vermiculite	- sheets

Thus, none of the mineral particles which contribute nearly all of the plasticity to natural clays are equidimensional; they are usually sheets or plates and sometimes rods or laths.

The other minerals (i.e., quartz, feldspars, carbonates, and oxides) common in natural clays are irregularly shaped, but their particles are usually not far from equidimensional. These particles can be considered, with reasonable accuracy, as rough-edged shapes approaching spheres.

B. Possible Arrangements of Particles

Since the sizes and shapes of particles in a natural clay vary, and especially since the most important particles are far from equidimensional, there are many possible arrangements of particles. Very irregular and very complex patterns are possible. The patterns of arrangements are less complex in compacted clays than in sedimentary clays, because stratifications are less important. Sedimentary clays usually have layers of different sized particles resulting from different conditions of deposition. Layering between the compaction lifts and variation of soil due to changes in the characteristics of borrow do, of course, occur. They are, however, of much less importance to structure than are the sedimentation stratifications in natural clays.

Mitchell (1956) worked out a semi-quantitative system for describing particle arrangement, or fabric. The items in his system are:

1. Texture
   a. Regularity of Components
   b. Transition between Components
2. Particle Orientation
   a. Within small areas, i.e., $< 1500\,\mu$

* Excellent electron photomicrographs of many specie of clay minerals are given in API (1951) and NAC-NRC (1955).

b. Within large areas, i.e., $> 1500\,\mu$
c. Clay around silt.

Mitchell used scales to describe these factors, as

a. Zero for no regularity to 90 for a high degree of regularity
b. Zero for poor transition to 90 for no discontinuities
c. Zero for perpendicular to 100 for parallel plates.

By means of a complete microscopic study, including measurements of extinction of thin sections under polarized light, Mitchell assigned numbers to the fabric of several clays. He enjoyed some success in relating his fabric measurements to engineering behavior; his technique is, however, too time-consuming and complex for routine work. Some of the results of Mitchell (1956) and Pacey (1956) will be used in later parts of this paper.*

In this paper descriptions of the arrangement component of structure will refer only to the orientation and spacing of adjacent particles. This simplification of Mitchell's complete description of particle orientation is justified, especially in compacted clays, since arrangement will be used as a concept only for qualitative purposes.

C. Arrangements of Particles Occurring in Soils

The arrangement of silt-sized particles can be, and has been, easily determined by means of a microscopic examination. Where there is convincing indirect evidence on the orientation of individual clay sized particles (a few microns and smaller), no direct measurements have been made. The preparation of clays for examination with an electron microscope in most techniques destroys the structure in a natural soil; in fact, some think that the severe pretreatment (especially drying) given specimens for the electron microscope actually changes the size and shape of some clay crystals, e.g., nontronite. Some of the replica techniques may give reliable electron photomicrographs of soil without disturbing the structure. The limitation of microscope resolution prevented Mitchell from making reliable determinations of particle orientation in zones smaller than 10 microns wide. He had to assume, therefore, that the orientations he measured within aggregates 10 microns and larger in size existed between individual particles. His work on specially prepared specimens of known particle orientation indicate his technique does give correct results. His and Pacey's data on natural soils are most likely correct.

In the preceding section, we saw that the particle orientation resulting from the flocculation of a sol approached a perpendicular array and that from the settling of a dispersed sol tended to a parallel array. Certainly non-salt flocculation gives a perpendicular orientation; salt flocculation gives a random orientation with individual particles tending to be parallel. The fact that a much denser deposit results from the sedimentation of a dispersed sediment than of a flocculated one proves that the particle packing in a dispersed sediment is much more efficient—i.e., in a more nearly parallel array.

* Using a technique similar to that of Mitchell's, Brewer and Haldane (1957) have made qualitative observations of clay particles when sands are permeated with clay suspensions. In general, the clay plates oriented themselves into positions parallel to the sand surfaces.

It has been pointed out that the soil engineer cannot neglect mass forces in the soil systems with which he works. In these soil systems, the terms "dispersed" and "flocculated" have slightly different meanings from those given them by the colloid chemist. To the soil engineer, a dispersed soil is one in which the net electrical forces between adjacent particles were repulsion at the time of deposition; a flocculated soil is one in which the electrical forces were attraction during deposition. This concept is very important.

When a deposit of dispersed soil is being formed by compaction or by sedimentation, the particles can slide past one another to positions giving a very efficient packing. Flocculating particles stick to one another when they approach each other; mechanical manipulation is much less effective in arranging particles.

Although the soil particles in a dispersed sediment are colloids, they are essentially in contact—not far apart as the term dispersed implies. The reasons for this have been presented: externally applied forces, as well as electrical forces, are important. The use of dispersion and flocculation in this sense, (which the colloid chemist might question) may not be wise. It is, however, widespread. As long as it is understood that a dispersed sediment implies interparticle electrical repulsion, and a flocculated sediment interparticle electrical attraction, the confusion should be minor.

Even though the particles in a flocculated sediment are nonparallel at the time of deposit formation, they can be made to assume a parallel array. The most obvious way to rearrange particles is through the application of a pressure. A parallel arrangement of particles in a flocculated deposit can result from one-dimensional consolidation under a very high pressure. Later sections discuss changes in particle orientation which can occur after deposition.

III. Nature of Water in Clay

The influence of water on the behavior of clay is tremendous. Terzaghi (1925) pointed out the importance of water in clay in his early work; nearly every investigator since has confirmed it. Of concern to the soil engineer are: amount of water; nature of water (how strongly held by the soil, electrolyte type and concentration, etc.); and stresses in the water. Even though much study has been devoted to clay water, knowledge of it is limited in extent and not well supported by experimental data.

Water is held to a clay particle by a force which decreases in magnitude as the distance from the particle surface increases. A rough approximation of the attractive force between soil and water as a function of distance is given by the electrical potential - distance plot, shown in Fig. 1. In other words, all of the water in the double layer is attracted to the soil particle. The force - distance plot is a smooth curve with no sharp discontinuities; it can be expanded and compressed by altering the system characteristics discussed in Section I or by altering the intergranular pressure.*

The soil-water attractive force consists basically of two components, namely:

1. Attraction of the dipolar water to the electrically charged soil.

* Applying an intergranular stress to adjacent colloids reduces the spacing of the colloids. The double layers are reduced.

2. Attraction of the dipolar water to the cations in the double layer; the cations are in turn attracted to the soil.

The mineral crystal carries net electrical charges, resulting from isomorphous substitution, and also local charges, resulting from the heterpolar bonds holding the mineral atoms together and from local discontinuities. Anion adsorption also contributes to the charge of the crystal. The water very near the soil is strongly held by these electrical charges on the soil. The water anywhere in the double layer is attracted by an induction type of force since the entire double layer is an electric field.

Further away from the colloid surface, the cations become fewer (see Fig. 1b) and the soil-cation linkage weaker. The force thus holding the cation hydration water to the soil is related to the potential in the double layer.

On the basis of the relative magnitude of force between water and soil, the water in a clay can be more or less arbitrarily divided into three types:* adsorbed water, double layer water and free water. The adsorbed water is that which is strongly held by the soil; the double layer water is all the water attracted to the soil; and free water is water which is not attracted to the soil at all. Adsorbed water is, therefore, the innermost part of the double layer water. While this classification of water is accepted by some, many others use "adsorbed" to describe all of the double layer water.

In Fig. 6 are shown, to scale, extremes of the most common plastic soil minerals, namely, kaolinite and montmorillonite. Fig. 7 shows approximately the extent of adsorbed and double layer water on these particles when they are isolated and have free access to water. (Each is to scale with its dimensions given. The scale in Fig. 7a is different from that in Fig. 7b.) This drawing

* Within the crystal lattices of some minerals exist OH^- groups. If these minerals are heated to 600 or 700° C, these OH^- groups undergo condensation polymerization; this reaction produces water. Since the OH^- groups, sometimes called "structural water", do not exist as water in the minerals, they are not considered here as "water".

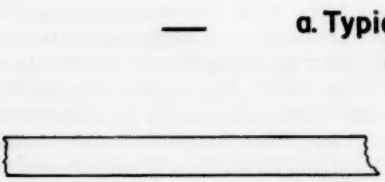

a. Typical Montmorillonite Particle
1000A by 10A thick
b. Typical Kaolinite Particle
1μ by .1μ thick
(10,000A by 1000A thick)

Fig. 6 — Clay Particles

Adsorbed Water
Double Layer Water
Kaolinite Crystal
10+A
4000A
10000A
10+A
4000A
a. Typical Kaolinite Particle 10,000A by 1000A
10 A
2000A
2000A
Montmorillonite Crystal
Adsorbed Water
Double Layer Water
b. Typical Montmorillonite Particle 1000A by 10A
CLAY WATER

FIG. 7

gives some idea of the amounts of tightly and loosely held water around the soil particles.

Heat of wetting data by Rosenqvist (1957), permeability data by Michaels and Lin (1955) and sorption data by Martin (1957) all show that the strongly-held water is only a few molecules thick, of the order of 10A. The force required to pull this adsorbed water off the mineral surface is prodigious—varying from 100 atmospheres for the outside water molecules to 10,000 atmospheres for the closest molecules. Since the charge density on kaolinite is about twice that on montmorillonite, there is a thicker layer of adsorbed water on kaolinite. The amount of adsorbed water expressed as a percentage of mineral weight is, however, much greater on montmorillonite since its specific surface is greater.

The adsorbed layer is about the same thickness as the Stern Layer in the Gouy-Chapman colloid theory. The concentration of exchangeable ions in this layer is very high, i.e., about 70% of the divalent exchangeable ions and 40% of the monovalent ions. Martin (1957) has described in detail the adsorbed layer on kaolinite; he gives the magnitudes of forces existing between water and soil and, the orientations and locations of the water molecules.

As with the adsorbed water, the double layer water is thicker on kaolinite than on montmorillonite because of the higher charge density on kaolinite. The dimensions given in Fig. 7 indicate that essentially all of the pore water in a clay under normal field conditions is within the double layer.

It is important to realize that the attractive forces in the double layer are forces required to pull water away from the soil. The force required to move water parallel to the sides of a soil particle is much less than the force required to remove, without replacement, the double layer water. When water flows through clay all but the adsorbed water moves, for example. This fact has led to the concept of double layer water as a "two-dimensional liquid". Another concept useful to the soil engineer is that of "mushy" particle for the soil plus its water. The "thickness of water film" obviously depends to a tremendous extent on the system characteristics and the intergranular pressure —and time. The movement of water nearest the surface of the particle takes considerable time since it is a diffusion process.

Since the double layer water is electrically attracted to the soil particle, its behavior is unlike that of "free" water. Many have postulated that the density, viscosity and other characteristics are different from normal water. Low (1958) has presented convincing experimental evidence that the density of the double layer decreases as the soil surface is approached. The effective viscosity of the double layer must be greater than free water. Unfortunately there are few experimental data (such as Low's) on the nature of clay water.

The controversy as to whether or not particles in a clay are in mineral to mineral contact has not yet been settled. The author's opinion on this question is summarized in the following paragraph.

Approaching clay particles first "contact" each other when their double layers interact. The change of interparticle spacing depends on the force system (as discussed in the following paper.) When the interparticle spacing has been reduced to a distance of approximately 20A, all of the water between the particles is tightly held. With sufficient externally applied effective stress and/or sufficient time and/or sufficient water removal (e.g., desiccation) much (all, in the case of desiccation) of the tightly held water can be extruded or removed. The surfaces of clay minerals usually contain rough spots which are greater than approximately 7A. Thus the extrusion by intergranular

pressure of all the interparticle water is impossible, although there might be isolated areas of mineral to mineral contact. The exchangeable ions also remain between approaching clay particles even though some of the water is forced out. The ions must remain to satisfy the net electrical charges on the particles.

Direct evidence on the nature of contact between clay particles is lacking. Bolt's consolidation tests (1955) on clays having essentially parallel particles showed that the average spacings between particles continued to decrease with an increase in pressure even at very high pressures (100 atms.). These data indicate the difficulty, if not the improbability, of extruding all of the water from between adjacent particles. The data in Fig. 8, however, do not show any significant sample volume decrease when the last of the adsorbed water was removed. Experiments by Rosenqvist and Fahn, et al, showed no change of clay sample volume upon the removal of water by freezing followed by sublimation. These data suggest that the last of the clay water was not at points of contact between particles.

There is a widely held concept that the cohesion in a natural clay is due to "water bonds". Water can, of course, resist tremendous tensile stress. The water-mineral link is strong, as has been stated. When soil water is put in tension by desiccation or by an externally applied force, an intergranular compressive stress results. In this situation, the water does contribute to soil strength. The fluid in the double layer plays a very important role in determining the interparticle forces, as colloidal theory shows. To go from these facts, however, to the belief that water bonds are the sole source of cohesion in clay is unwarranted.

The limitations of the water-bond concept are indicated by the fact that the maximum cohesion a clay ever possesses occurs when the clay is dry, i.e., contains no water.

Presented in Fig. 8 are the results of a series of tests on remolded Boston blue clay which show that an increase in strength occurs with higher drying temperatures. One cannot attribute the strength of the dry clay to "adsorbed moisture" since all of the adsorbed moisture is gone at 200^{0} C. Soil moisture should be considered a "weakener" or "lubricant" of clay rather than a "glue".

In summary, water has an important effect on the properties of clay. A great deal remains to be determined about clay water, especially the magnitudes of the pressures existing in the water and the effects of these pressures on the behavior of the clay. In the average natural clay which the engineer encounters, essentially all of the water is within the double layer. Any classification of the clay water is somewhat arbitrary and will present difficulties because the nature of soil water depends on variable properties of the soil-water system.

IV. Theory of Compaction

A. Compaction Process

An explanation will now be given for the characteristic plot of molding water content versus compacted density shown in Fig. 9. The lower plot is for a lower compactive effort than the upper curve, i.e., the lower could represent standard and the upper modified AASHO compaction effort.

First, however, the concept of "water deficiency" needs to be

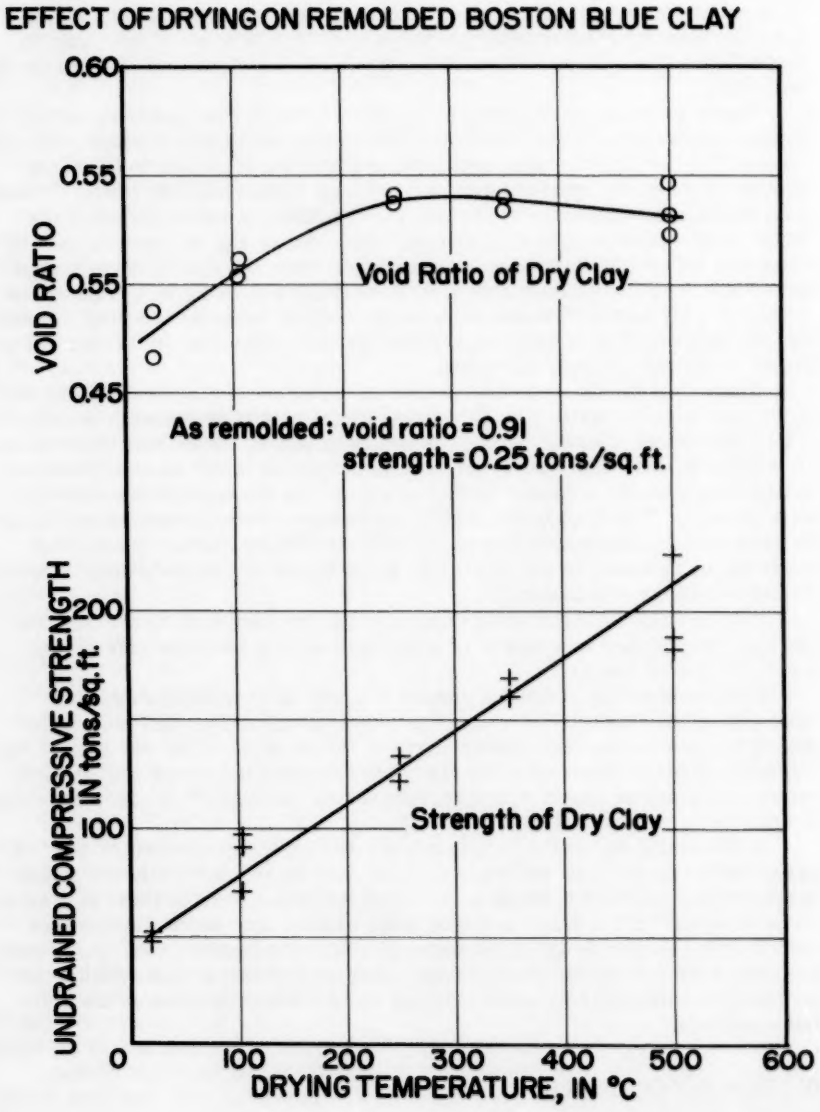


Fig. 8 — Strength of Dried Boston Blue Clay

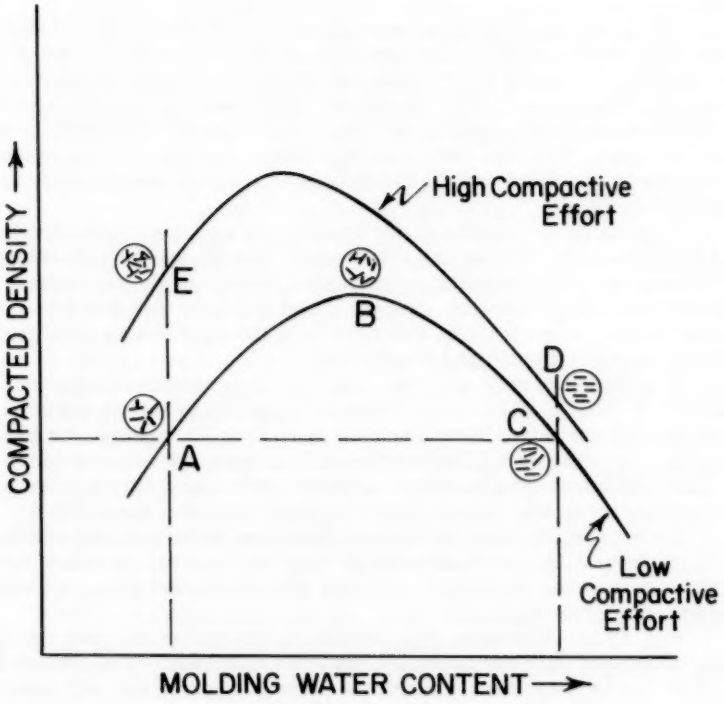


Fig. 9 — Effects of Compaction on Structure

mentioned. This concept recognizes that any given soil particle under any given state of stresses requires a certain amount of water to develop fully its double layer. The difference between this needed water and the existing water is deficient water which the particle will try to imbibe. The amount of water used in clay compaction is almost always less than that wanted by the soil; nearly all compacted plastic soils in the as-molded state have a deficiency in water.

At w_A in Fig. 9 there is not sufficient water for the diffuse double layers of the soil colloids to develop fully. Actually the small amount of water present at A gives a very high concentration of electrolyte which depresses the double layer. The double layer depression reduces the interparticle repulsion, thereby causing a tendency toward flocculation of the colloids. As has already been noted, flocculation generally means a low degree of particle orientation and low density. The low density and random particle array is illustrated in Fig. 9.

Increasing the molding water from w_A to w_B expands the double layers around the soil particles and also reduces the electrolyte concentration. The reduced degree of flocculation permits a more orderly arrangement of particles and a higher density. The term "lubrication" well describes the increased interparticle repulsion which permits the particles to slide past each other into a more oriented and denser bed.

A further increase of water from w_B to w_C results in a further expansion of the double layer and a continued reduction in the net attractive forces between particles. Even though a more orderly arrangement of particles exist at w_C than at w_B, the density of C is lower because the added water has diluted the concentration of soil particles per volume (there is not a marked decrease in air content from B to C as there was from A to B).

Because of the water deficiency, the water present is subjected to tension as the colloids try to draw water from the outside. Because there is less water at w_A than w_B or w_C, the water deficiency, and thus pore water tension is greater at w_A.

Fig. 9 also illustrates what happens to soil structure when the compactive effort, and thus compacted density, are increased. The greater the input of work, the more nearly parallel are the clay particles, and, usually, the closer together they are. At high molding water contents (and high degrees of saturation), increased compactive effort may merely align particles without significantly altering the particle spacing.

The magnitude of the variation of structure with molding water depends on the particular soil. Some soils will show an extreme variation, going from a random to parallel orientation, as is illustrated in Fig. 9. Other soils will exhibit only slight improvement of orientation with increased molding water. This slight variation arises with soils which: 1) are fairly well dispersed dry of optimum or 2) are still partly flocculated wet of optimum. For these soils, there is an improvement in orientation with increased molding water; but the improvement is less than the extreme case shown in Fig. 9.

B. Variables Which Can Be Altered by the Soil Engineer During Compaction

The theory presented in this paper indicates that the compaction process and, therefore, the properties of compacted soil depend on the seven system variables discussed in Section I and also on other factors to be discussed in Section V. Many of these variables cannot feasibly be altered or

controlled by the soil engineer on a given job; in such cases, he can use the theory to predict the effects of changes brought about by nature.

The soil engineer can, however, alter several variables during a given set of boundary conditions. These variables are:

1. Type of Compaction
   Little is known on how the type of compaction influences the fundamental components of structure listed in the preceding section. Dynamic compaction gives a different product from static compaction (e.g., see ASCE (1950), Seed, Lundgren and Chan (1954), and Seed and Monismith (1954)); how and why are not clearly understood. The time and area of contact between compacting element and soil are apparently important variables.
2. Amount of Compaction
   For any method of compaction and any clay at a given water content, the greater the compactive work employed, the greater is the density (particles closer together) and the better usually, the particle orientation that occur. Although increased compactive effort usually gives greater strength, but it can give lower strength. This is well known, of course.
3. Amount of Water
   The more the water added, other variables being constant, the greater is the repulsion between particles. Thus water weakens, or "lubricates," clay.
4. Additives
   The rheology of a soil-water-air system being compacted can be sharply changed with additives. Dispersants, in effect acting as additional water, reduce net interparticle attractive forces. These chemicals reduce optimum water content and increase maximum compacted density. Aggregants do the reverse of dispersants; by increasing flocculation, they raise optimum water content and reduce the maximum density. Dispersants permit a higher degree of particle orientation, while aggregants result in a more random array. Other types of additives can affect the compaction process.

C. Measurements of Structure

Using Mitchell's technique, Pacey (1956) measured particle orientation as a function of molding water content on compacted Boston blue clay. His data presented in Fig. 10* agree with our theory, namely: the more the molding water and the greater the density, the more nearly parallel are the clay particles.

Direct measurements (as made by Mitchell and Pacey) of particle orientations within a clay are valuable data. Since such measurements are, unfortunately, difficult to take, a simple method of determining particle orientation is greatly needed. Possibly the amount of shrinkage upon drying could be used as a measure of average orientation. Any given soil with parallel particles should undergo more volume reduction upon drying than the same soil with its particles in a random array.

* Pacey ran his tests in a miniature mold. Curves marked b in Fig. 10 were obtained by using a compactive effort approximating standard AASHO and a, by using modified AASHO.

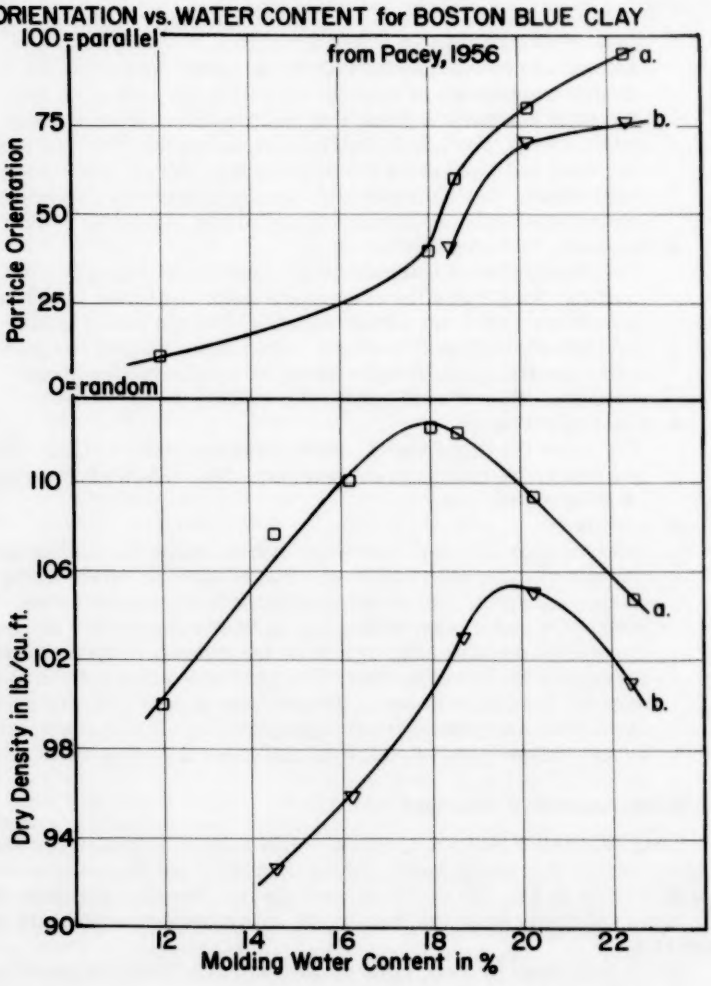


Fig. 10 — Orientation vs Water Content for Boston Blue Clay

Fig. 11 shows the effect of oven drying on the void ratio as measured by Pacey on Boston blue clay (shrinkage of the compacted soil represented by curve a in Fig. 10). Fig. 12 shows shrinkage data obtained by Tiles (1956) on compacted kaolinite. In Fig. 13 are plotted Pacey's data to show the relationship between particle orientation at the start of drying vs the amount of shrinkage upon drying. These figures (11, 12 and 13) show that the more nearly parallel the particles are, the greater the shrinkage of the soil upon drying.

A slaking test may give some qualitative indications of the electrical forces between particles. A clay in which the individual particles possess high net attractive electrical forces would resist water attack better than one in which the particles have net repulsive forces between them. The hope that a slaking test might be a useful structure indicator was supported by the fact that a sample of dispersed clay was found to slake before a sample of the same clay having a flocculated structure. Interpretation of slaking tests can be complicated by the effect of structure on permeability. Water gets into a dispersed sample at a slower rate than it does into a flocculated sample because of the lower permeability of the dispersed sample.

An indication of the nature of the electrical forces in a clay can also be obtained by measuring the pressure required to prevent swelling of the clay when it is put in contact with water. The pressure, i.e., "swell pressure," may be a useful way of characterizing the force component of structure.

Hvorsley (1938) dried samples which had been subjected to different loading conditions. He found that the pattern of cracking on drying tended to be perpendicular to the direction of force application, i.e., the cracks were parallel to particle orientation.

Haggarty (1957) studied the relationship between particle orientation and shrinkage strains of a clay upon drying. He found that: 1) a clay with oriented particles shrinks the most in a direction perpendicular to the plates and least parallel to the plates and 2) a clay with randomly oriented particles shrinks equally in all directions.

The facts cited above indicate that a simple shrinkage-slaking test can give considerable information on the structure of a clay.

V. Changes in Soil Structure

The engineering behavior of a given clay is determined by the structure which the clay has at the time the behavior is measured. This final structure depends on both the initial structure and on the changes which have occurred to the initial structure. In other words, the properties of a given clay are developed in the following sequence:

1. Placement or deposition conditions determine the initial soil structure.
2. The initial soil structure is changed to give the final structure.

This development is shown in Fig. 14.

Earlier portions of this paper discussed the nature of structure and the effects of placement conditions on it. Sedimentary clay is shown on Fig. 14 to make the picture more nearly complete. A discussion of deposition variables is outside the scope of this paper. This section discusses the causes and effects of changes in soil structure.

Normal pressure tends to push particles together, i.e., decrease interparticle spacing, and, as has already been noted, align particles. Shear

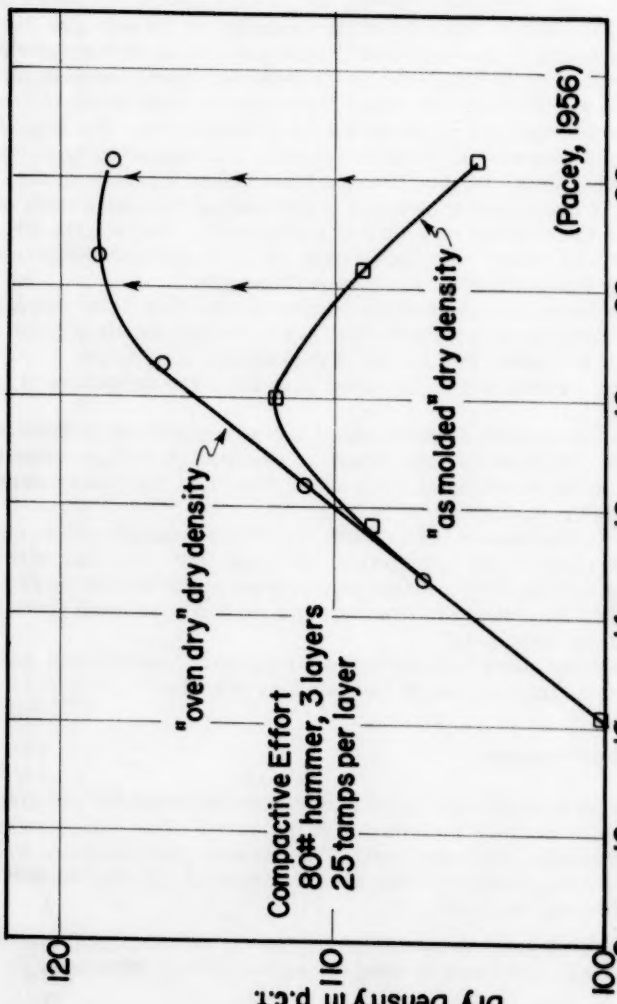


Fig. 11 — Dry Density vs Molding Water Content for Boston Blue Clay

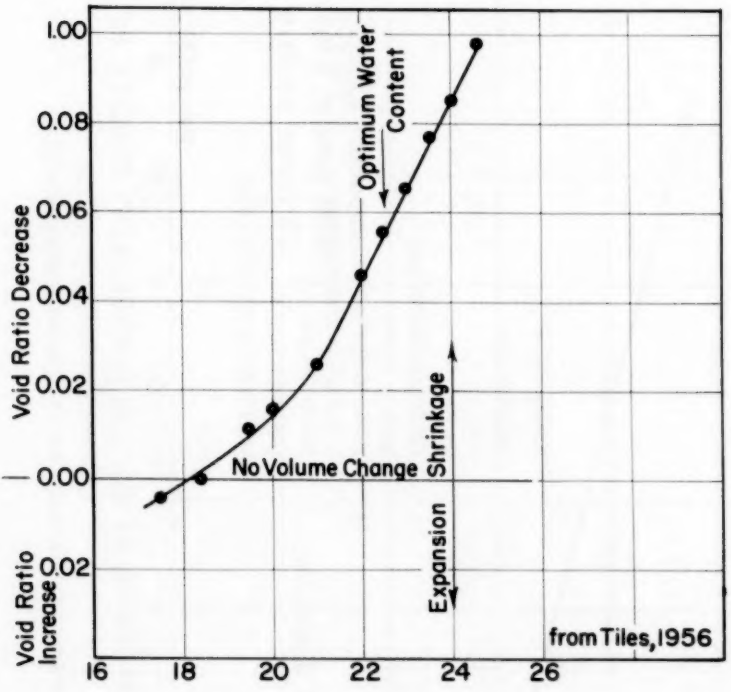


Fig. 12 — Effect of Oven Drying on Void Ratio of Compacted Kaolinite

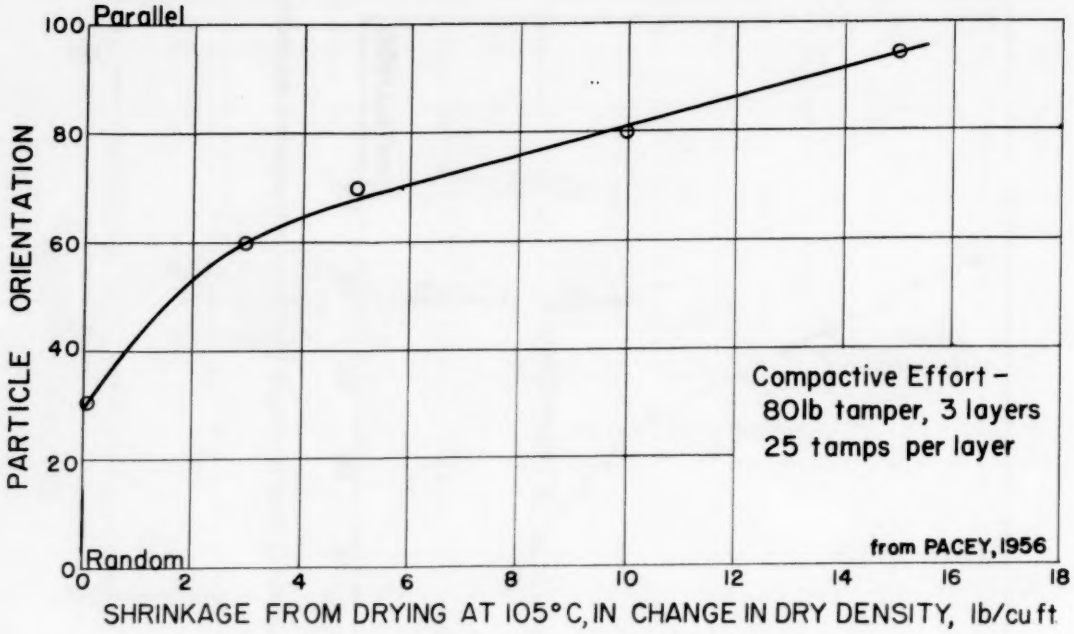


Fig. 13 – Orientation vs Shrinkage for Compacted Boston Blue Clay

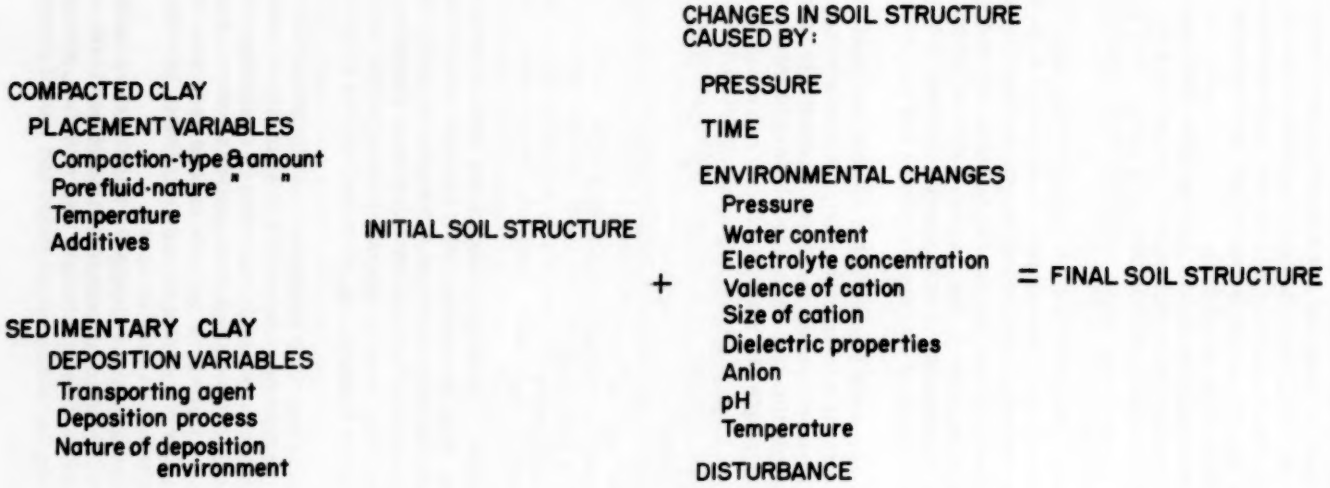


Fig. 14 — Development of Soil Structure

deformations also tend to align them. One-dimensional compression is much more effective than three-dimensional compression in pushing the particles toward a parallel orientation. Time should be considered with pressure since they usually go hand-in-hand. Not only is time required for the adsorbed water to be extruded, but also for the layers of adsorbed water of adjacent particles to slide in relation to each other.

Seed et al. (1958) showed dramatically that repetitive load applications caused a significant increase in the strength of a compacted silty clay. This strength increase was in addition to that caused by aging. Seed attributed the strength increase to an alteration in soil structure. This explanation certainly seems to be the correct one.

Another factor which alters structure is environmental changes, under which are grouped alterations in the boundary conditions of a deposit and characteristics of the deposit which depend directly on these boundary conditions. The colloid theory states that the interparticle forces depend on second through ninth items listed in Fig. 14; we must also include changes in intergranular stress. A reduction in intergranular pressure lets in water between particles. This causes an increase in volume and a decrease in strength.

Any change in the soil-water system which expands the double layer tends to decrease soil strength (at a given void ratio), since interference by the double layers of two adjacent colloids increases interparticle repulsion. This concept leads to the prediction that any of the following changes would generally reduce the shear strength of a clay:

1. Reduction of electrolyte concentration
2. Cation exchange from high to low valence (e.g., Ca^{++} to Na^{+})
3. Adsorption of anions (e.g., phosphate)
4. Exchange from a cation of small hydrated radius to a cation of large hydrated radius (e.g., Na^{+} to Li^{+})
5. Increase of dielectric constant of pore fluid
6. Increase of pH of pore fluid
7. Decrease of temperature
8. Increase in water content.

The influence of some of these eight changes has been investigated. The Norwegians (Bjerrum, 1954; Rosenqvist, 1955; Bjerrum and Rosenqvist, 1957) have, for example, conclusively shown that a reduction of electrolyte by leaching can reduce both the undisturbed and remolded shear strengths. The reduction of strength resulting from an increase in moisture is well known. The effect of some of the other changes, e.g., temperature, has yet to be thoroughly investigated.

Disturbance can change particle arrangement and interparticle forces. Particles which have been pushed into contact by years or, of more importance, eons of high pressure can be separated by remolding or even less severe disturbance, e.g., deformations from applied stresses. Experiments by ceramists have shown that extrusion tends to align particles. Mitchell (1956) proved with measurements that not only remolding, but even shear strains, arranges particles in a parallel array.

There are several results of the changes in soil structure listed in Fig. 14 and described above which are of considerable importance to the soil engineer. Among these are sensitivity, strength loss with time, strength gain with time (thixotropy), creep, secondary compression and loss of strength with strain past peak strength.

Sensitivity is caused by a structure change on remolding. In other words, the structure which will be formed by a thorough working (which will thus determine the remolded strength) must be unlike that which existed prior to the disturbance (which will determine the undisturbed strength). A change of the structure of the clay from what it was at deposit formation to what it is at time of test is necessary for the clay to be sensitive. Factors which can cause such a structure change fall into two groups:

1. Time and Pressure which push particles into alignments and spacings not obtainable by remolding.
2. Environmental changes from the time of deposition to the time of test which tend to increase interparticle repulsion—i.e., the eight listed above.

The three factors most important in developing sensitivity in natural clays are probably time, pressure and the reduction of electrolyte concentration in the pore fluid.* Any of the factors, however, can cause sensitivity. This fact is emphasized by the following test data on Boston blue clay:

	Undisturbed Strength, in psf.	Remolded Strength, in psf.
Clay leached with water	450	41
Clay leached with $Na_6P_4O_{13}$	220	0

Leaching the saturated clay (at constant volume) with the sodium polyphosphate permitted ion exchange Na^+ for Ca^{++}, and anion adsorption, i.e., phosphate. These two factors changed the sensitivity from eleven to infinity. This tremendous increase in sensitivity occurred even though there was an increase in electrolyte concentration and little change in the other factors.

The reduction of electrolyte concentration is very widespread since it has occurred in nearly all natural clays laid down in a marine environment. (The average marine clay was originally deposited in water containing about 35 grams of salt per liter and the clay water contains, after leaching with ground water, about 5 grams of salt per liter). This is of little importance to the development of sensitivity of compacted clays. The important factors for man-placed clays are probably water pick-up, time and pressure. Samples compacted dry of optimum have structures which are more greatly changed than those of wet compacted samples. Sensitivity is much more likely, therefore, in clay compacted dry of optimum than it is in clay compacted wet of optimum.

Strength loss with time can be caused by an increase in moisture which increases interparticle repulsion. The increase in moisture could be the result of a change in ground water conditions or a reduction in intergranular pressure. The leaching data on the Boston blue clay, showing a 50% reduction in undisturbed strength, indicates that significant strength loss can occur from changes in environment. Disturbance can also be a source of strength loss. In compacted clays, however, especially those compacted dry of optimum, moisture increase would appear to be the most important cause of strength loss. Strength loss with time usually implies an increase in sensitivity.

* Well known research at the Norwegian Geotechnical Institute (Publications Nos. 4, 9, and 25) has proved the importance of electrolyte reduction in producing sensitivity.

Strength gain with time results from the same factors as those causing strength loss, except that the changes are in the opposite direction. Pressure, time, moisture reduction, electrolyte increase, ion exchange from low to high valence, pH decrease, temperature increase, etc. all strengthen soil. Thixotropy experiments have been conducted in which attempts have been made to hold all factors other than time constant. The strength gains measured can be explained solely by the time required for particles to arrange themselves into an equilibrium array under even the small pressures acting. Imperfect control of the other factors, however, (moisture decrease, ion exchange, temperature change, etc.) may well have had an effect on the soil strengths measured.

Creep probably results from particle realignment and greater interparticle spacing. We have already noted that strains align particles. Deformations tend to push particles apart at points of contact, permitting water to enter. Clay sheared in triaxial tests in which the water content of the entire specimen was held constant have shown an increase in moisture content in the failure zone and a decrease in the end or "dead" zones.

Secondary Compression results from changes in particle orientations and spacings which occur as tightly held water molecules on one particle are slid past those on another particle or as some of this water is extruded from contact areas.

Loss of strength past peak strength is caused by changes in orientations and spacings of soil particles. This loss of strength will, of course, be larger in those soils which have structures sensitive to disturbance.

Because of the similarities in causes, the following results of changes in structure might well go together: sensitivity, creep, and loss of strength with strain past peak strength.

The discussion in this section applies in general to both naturally and man-made deposits. The factors which are important to the development of properties in compacted clays, however, are different from those in natural clays. In the following sections are presented the effects of some of these factors on compacted clays.

Summary and Conclusions

This paper describes the nature of clay structure, a term referring to the arrangement of particles and the electrical forces acting between them. The effects of forces and environmental factors on structure are inferred from the principles of colloid chemistry and crystal chemistry.

The theoretical considerations presented herein throw considerable light on the fundamental nature of clay-water systems. These considerations plus available experimental data fall short, however, of furnishing a complete understanding of clay-water systems. Some of the topics which need study are noted in the following paper.

REFERENCES

1. API, "Reference Clay Minerals," Research Project 49, Columbia Un., NYC, 1951.
2. ASCE, "Development of CBR Flexible Pavement Design Method for Airfields," A Symposium, ASCE Trans. Vol. 115, p. 453, 1950.

3. Aitchison, M. E., "The Nature, Extent and Engineering Significance of the Conditions of Unsaturation in Soils Within the Australian Environment," Ph.D. Thesis, Univ. of Melbourne, 1956.

4. Bjerrum, L., "Geotechnical Properties of Norwegian Marine Clays," Publication No. 4, Norwegian Geotechnical Institute, Oslo, 1954.

5. Bjerrum, L. and Rosenqvist, I. Th., "Some Experiments with Artificially Sedimented Clays," Publication No. 25, Norwegian Geotechnical Institute, Oslo, 1957.

6. Bolt, G. H., "Analysis of the Validity of the Gouy-Chapman Theory of the Electric Double Layer," Jour. Colloid Science 10:206, 1955.

7. Bolt, G. H., "Physical-Chemical Analysis of the Compressibility of Pure Clay," Geo. Technique, Vol. 6, No. 2, pp. 86-93, June, 1956.

8. Bolt, G. H., and Miller, R. D., "Compression Studies of Illite Suspensions," Soil Science Proc. 19:285-288, 1955.

9. Brewer, R. and Haldane, A. D., "Preliminary Experiments in the Development of Clay Orientation in Soils," Soil Science, No. 4, Vol. 84, October, 1957.

10. Casagrande, A., "The Structure of Clay and Its Importance in Foundation Engineering," Journal of BSCE, April, 1932.

11. Chapman, D. L., Phil. Mag. (6) 26 (1913).

12. Evans, R. C., Crystal Chemistry, Cambridge University Press, 1948.

13. Fahn, R., Weiss, A., and Hofmann, U., "Thixotropy in Clays," Berichte der Deutsche Keramischen Gesellschaft, 30, p. 21, 1953.

14. Gouy, G., J. Phys. (4) 9 (1910).

15. Haggarty, G. J., "Measurement of Shrinkage Strains in Clays," S.M. Thesis, M. I. T., 1957.

16. Hvorslev, Juul, "The Shearing Resistance of Remolded Cohesive Soils," Proc. Soils and Foundation Conference, U. S. Engr. Dept., Boston, Mass., June, 1938.

17. Kruyt, H. R., Colloid Science, I, Irreversible Systems, Elsevier Publishing Co., New York, p. 195, 1952.

18. Ladd, C. C., "Swelling of Compacted Clay," S. M. Thesis, M. I. T., 1957.

19. Lambe, T. William, "Structure of Inorganic Soil," ASCE Separate 315, October, 1953.

20. Lambe, T. William, "The Permeability of Compacted Fine Grained Soils," ASTM, 1955.

21. Low, Philip F., "Movement and Equilibrium of Water in Soil Systems as Affected by Soil-Water Forces," Highway Research Board Meeting, January, 1958.

22. Martin, R. T., "Water Vapor Sorption on Kaolinite: Hysteresis," Clays and Clay Minerals, N. A. S. - N. R. C., In Press, 1957.

23. Michaels, Alan S., and Lin, C. S., "Effects of Counter Electro-Osmosis and Sodium Ion Exchange on the Permeability of Kaolinite," Industry and Engineering Chemistry, 1955.

24. Michaels, Alan S., "Mechanism of Deflocculation of Clays by Condensed Alkali Phosphates," Ind. Eng. Chem., In Press, 1957.

25. Mitchell, James K., "The Importance of Structure to the Engineering Behavior of Clay," Sc.D. Thesis, M. I. T., 1956.

26. National Academy of Science and National Research Council, Clays and Clay Minerals, Publication 395, Washington, 1955.

27. Pacey, John G., Jr., "The Structure of Compacted Soils," S.M. Thesis, M. I. T., 1956.

28. Pauling, Linus, "The Nature of the Chemical Bond," Cornell University Press, 1951.

29. Rice, Francis Owen, and Teller, Edward, "The Structure of Matter," Wiley, 1949.

30. Rosenqvist, I. Th., "Investigations in the Clay-Electrolyte-Water System," Publication No. 9, Norwegian Geotechnical Institute, Oslo, 1955.

31. Rosenqvist, I. Th., 1957 Proc. of Fourth International Soils Conference.

32. Schofield, R. K. and Samson, H. R., "The Deflocculation of Kaolinite Suspensions and the Accompanying Change-Over from Positive to Negative Chloride Adsorption," Clay Minerals Bulletin, Vol. 2, No. 9, British Mineralogical Society, July, 1953.

33. Schofield, R. K. and Samson, H. R., "Flocculations of Kaolinite Due to the Attraction of Oppositely Charged Crystal Faces," Discussion of the Faraday Society, No. 18, 1954.

34. Seed, H. B., Lundgren, Raymond, and Chan, Clarence K., "The Effect of Compaction Method on the Stability and Swell Pressure Characteristics of Soil," Highway Research Board, 1954.

35. Seed, H. B., McNeil, R. L., and de Guenin, J., "Effects of Repeated Loading on Strength and Deformation Characteristics of Compacted Clay," presented at Chicago ASCE Meeting, Feb., 1958.

36. Seed, H. B. and Monismith, Carl L., "Some Relationships Between Density and Stability of Subgrade Soils," Highway Research Board, 1954.

37. Terzaghi, Karl, Erdbaumechanik, Vienna, F. Deuticke, 1925.

38. Theissen, P. A., "Wechselseitige Adsorption von Kolloiden," Zeitschrift fur Elektronchemie, Vol. 48, p. 675, 1942.

39. Tiles, I., "Shrinkage and Strength of Compacted Sand-Clay Mixtures," S. M. Thesis, M. I. T., 1956.

40. van Olphen, H., "The Stabilization of Montmorillonite Sols by Chemical Treatment," Recueil Des Travaux Chimiques Des Pays-Bas, Amsterdam, Sept. and Oct., 1950.

Paper 1655 May, 1958 SM 2

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

THE ENGINEERING BEHAVIOR OF COMPACTED CLAY[a]

T. William Lambe,[1] M. ASCE
(Proc. Paper 1655)

SYNOPSIS

This paper employs the principles of colloid chemistry and crystal chemistry to give a physical explanation of the engineering behavior of compacted clay. The relationships between clay structure and behavior are described and then illustrated with experimental data.

INTRODUCTION

This paper is the second of a series of two dealing with the structure of compacted clay. The first paper presented the theoretical principles needed to describe structure and to predict the factors affecting it. In this paper these principles are employed to give a physical explanation of the engineering behavior of clay.

As the careful reader will discover, the concepts are often neither as definite nor as complete as he would like. Improvements (and probably corrections) to these concepts will result from research on the fundamentals of clay behavior that is being conducted in a number of soil laboratories. From both soil scientists and soil engineers contributions to the understanding of clay properties can be anticipated.

I. Permeability of Compacted Clay

A. Importance of Permeability Studies on Clay

The permeability of a soil is one of its three important and fundamental

Note: Discussion open until October 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1655 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 2, May, 1958.

a. Paper presented at a meeting of the ASCE, Chicago, Ill., February, 1958.

1. Director, Soil Stabilization Lab., and Assoc. Prof. of Soil Mechanics, Massachusetts Inst. of Technology, Cambridge, Mass.

properties, compressibility and strength being the other two. Permeability enters into problems of seepage, settlement and stability. In the core of an earth dam, for example, the rate of leakage, rate of settlement and rate of strength build-up all vary directly with the permeability.

Permeability studies on clays not only are valuable for use in practical engineering problems, but also are helpful in fundamental research on the nature of clays. Michaels and Lin (1955) used permeability tests to study the nature of water in kaolinites; Olsen (for his Sc.D. thesis at M. I. T.) is currently studying clay structure with permeability measurements.

B. Factors Affecting Permeability

In an earlier paper the author (1955) discussed the factors affecting the permeability of fine grained soils. The factors treated were:

Soil composition
Characteristics of permeant
Void ratio
Degree of saturation
Structure.

For a given soil and permeant, i.e., first two factors constant, the relationship between permeability and the other factors is expressed by the Kozeny-Carman equation,

$$K = \frac{1}{k_o S^2} \frac{e^3}{1+e}$$

where,

K = absolute permeability
k_o = constant depending on pore shape and tortuosity of flow
S = specific surface
e = void ratio.

The Kozeny-Carman equation does not consider variations in degree of saturation.

A liberal interpretation of the Kozeny-Carman equation might make $k_o S^2$ a term indicating the tortuosity of flow or actually the orientation of soil particles. This orientation component of structure is the important component as far as permeability is concerned. The orientation term is,

$$k_o S^2 = \frac{1}{K} \frac{e^3}{1+e} .$$

Previous work (Michaels and Lin, 1954; Lambe, 1955) has proved that the structure of a soil has a very large effect on its permeability. The following section discusses the effect of the structure of compacted clay on permeability.

C. Effect of Structure on Permeability

Test data from permeability tests on two soils will be used to describe the effect of structure on permeability. The data, shown in Figs. 1 and 2, were obtained on samples first compacted and then subjected to flow until a

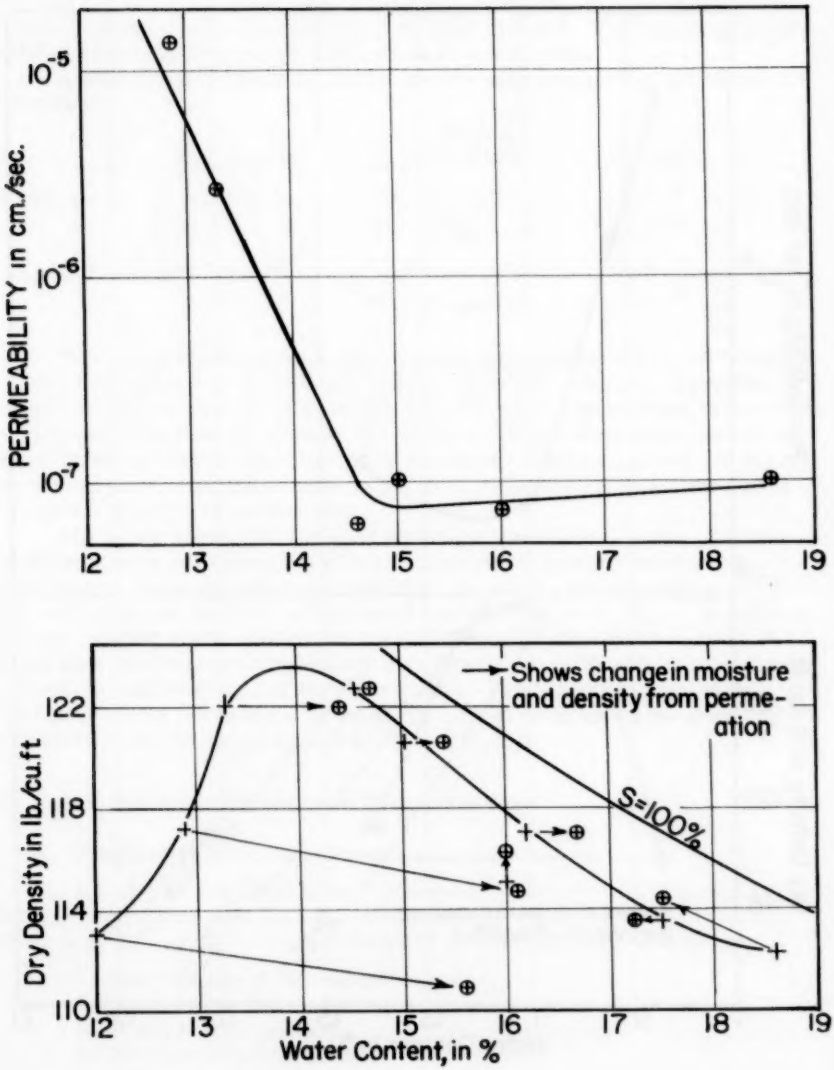


Fig. 1 — Compaction-Permeability Tests on Jamaica Sandy Clay

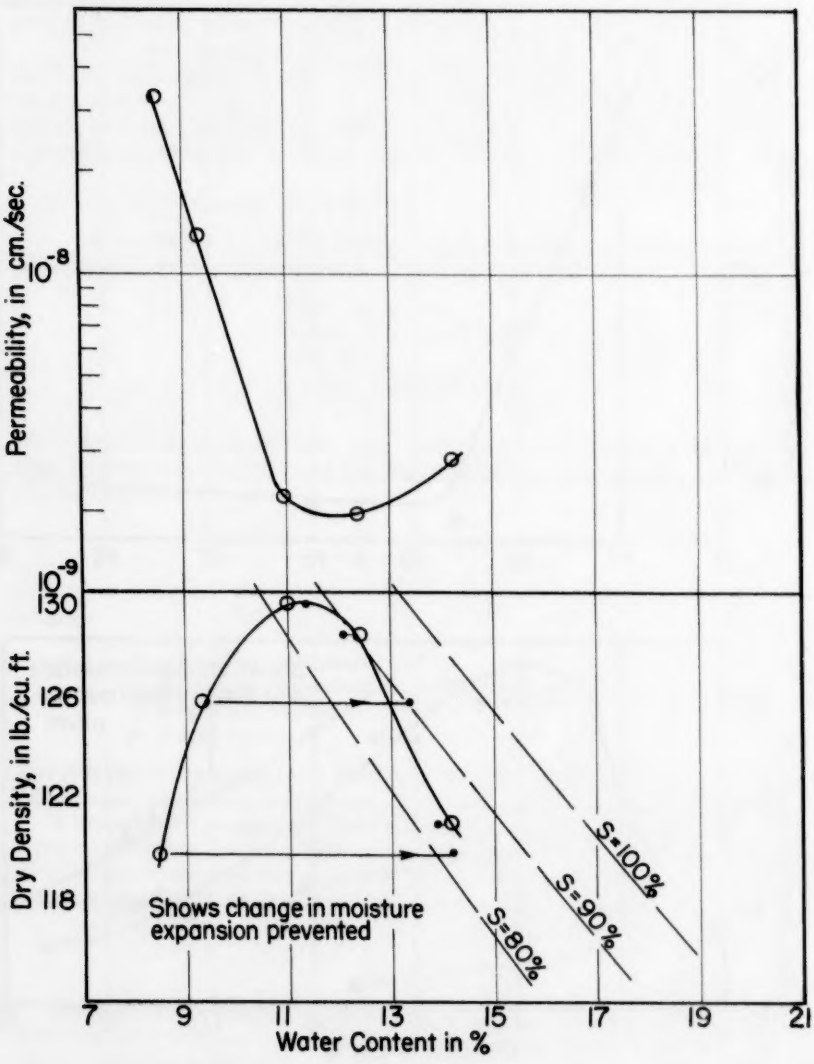


Fig. 2 — Compaction-Permeability Tests on Siburua Clay

steady-state case existed. The tests on the Jamaica soil (Fig. 1) were run in such a way that the soil could change volume during permeation; the Siburua clay was permitted to densify but not expand. Changes in moisture and density that occurred during flow are shown in Figs. 1 and 2. The values of permeabilities plotted in Figs. 1 and 2 are those at equilibrium.

A comparison of permeabilities for dry-side compaction with those for wet-side is:

Soil	Dry Density	Molding Moisture	Permeability
Jamaica Soil	116 lb/cu.ft.	Dry of optimum	1.4×10^{-5} cm/sec.
	116	Wet of optimum	8×10^{-8}
Siburua Clay	122 lb/cu.ft.	Dry of optimum	3×10^{-8}
	122	Wet of optimum	2.5×10^{-9}

This comparison shows a much higher permeability for a soil compacted dry of optimum than for the soil compacted wet of optimum. Since the sample compacted dry of optimum has a more random orientation, it has more large pores than does the sample with more nearly parallel arrangement obtained from wet-side compaction. The larger the individual pores for any given total pore area, the greater is the flow, since the permeability varies as a power function of opening size.

Olsen has shown that the $k_o S^2$ structure term in the Kozeny-Carman equation gives a good measure of soil structure or, particle orientation. In other words, the more nearly parallel the clay plates, the smaller is the average void available for flow and the more tortuous the flow. Fig. 3 is a plot of $k_o S^2$ vs molding water content for the data (Fig. 2) on the Siburua clay. It shows that the reduced permeability with increased molding moisture content can well be explained by particle orientation. The larger the repulsive electrical forces between particles at the time of compaction the lower the permeability of the resulting soil is.

II. Compression and Expansion of Compacted Clay

A. Components of Volume Change

If a clay is subjected to an "all around" pressure with drainage permitted, it compresses; upon load release, it usually expands. These volume changes in clay are due to one or more of the five following components:

1. Gas—change of volume or amount
2. Particle Diminution
3. Particle Deformation
4. Particle Rearrangement
5. Change in Size of Micelle.

Gas can contribute to a volume decrease in a soil either by compressing, because of an increase in pressure, or by going into solution in the pore water because of a greater pressure or a lower temperature. On the other hand, it can contribute to a volume increase by expanding under a load release or by coming out of solution of the pore fluid when the pressure is reduced or the temperature raised. Volume changes of fine-grained soils caused by gas

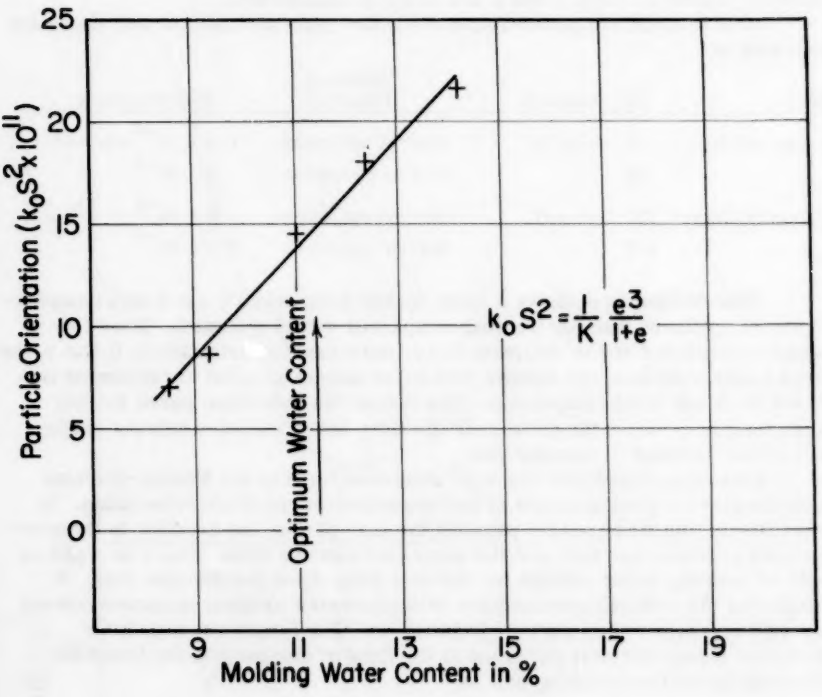


Fig. 3 — Particle Orientation vs Molding Water for Siburua Clay

are complex and not well understood. This component will be excluded from the following discussion of volume change.

While Components 2 and 3 (particle diminution and particle deformation) undoubtedly occur in certain soils, they are probably insignificant in clays. The contact area in granular soils is so small that the usual stresses are large enough to crush unsound mineral (or rock) aggregates or to deform them elastically. The particles of such friable materials as caliche or weathered granite often decrease in size during laboratory or field compactions. The amount and type of contact area in clays, however, is such that particle crushing or elastic deformation from the stresses which the soil engineer would normally apply to the soil is probably not measurable.

Particle deformations can, however, make a significant contribution to volume changes in a soil composed of large plate-like particles. Loading and unloading a dry mass of sand-size mica flakes demonstrates this. To use this experiment to deduce the mechanism of clay compression is not wise, as shown by the test results plotted in Fig. 4. In the test a sample of dry mica

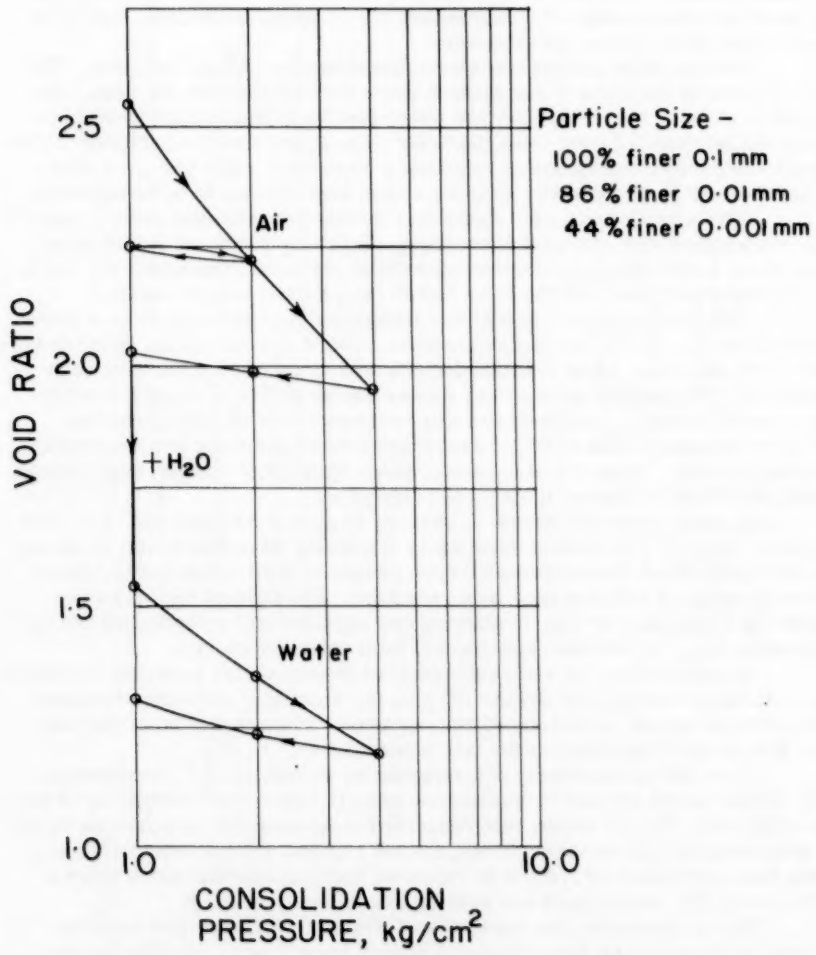


Fig. 4 — Compression of Muscovite

(muscovite) having silt- and clay-size particles was loaded and unloaded, leached with water, and finally loaded and unloaded again. The results show: 1) a marked decrease in sample volume upon contact with water and, 2) a larger rebound in the mica-water system than in the mica-air system. Expressed as a percentage of compression, the rebound of the mica-water is more than twice that of the mica-air.

Both of these actions are characteristics of colloidal behavior. The water permits the mica flakes to pack more densely than the air does. Because it is more polar than air, the water results in greater electrical repulsions between adjacent mica particles, i.e., lubricates the particles. Since there are greater interparticle repulsions when water is in the mica pores than when air is, the greater expansion upon load release is to be expected.

This compression test shows that merely reducing the particle size of the mica introduces colloidal phenomena. With clay particles (which have relatively large electrical surface potentials) colloidal phenomena are much more important than with the mica flakes (which have low potentials.)

The rearrangement of soil particles from a random array to a more orderly array contributes to compression. There are convincing data (Hvorslev, 1938; Mitchell, 1956) that one dimensional compression tends to align particles. The sudden decrease in volume shown in Fig. 4, when the water permeated the mica sample is due to a rearrangement of mica particles. Volume decreases caused by the rearrangement of particles are essentially nonrecoverable. When the volume increases from other causes, some minor changes, of course, occur in particle orientation.

In most clays Component 5, changes in size of soil micelle—i.e., soil mineral particle plus double layer water (including adsorbed water, is an important contributor to compression upon pressure application and is almost the sole cause of rebound upon load reduction. The influence of this component is illustrated by Fig. 5, showing two adjacent soil colloids and the relationship (Fig. 5c) between spacing and total potential energy.

At equilibrium the externally derived intergranular pressure (consider the colloids as having unit areas) $\bar{\sigma}$ plus the electrical attraction between the particles equals the electrostatic repulsion. The spacing 2d is the one that brings about equilibrium for this force system.

If we add an increment of intergranular pressure, $\Delta\bar{\sigma}$, as shown in Fig. 5b the forces are not in equilibrium until the inter-colloid spacing is reduced by Δ2d. Fig. 5c shows that reducing the spacing Δ2d requires an input of work equal to ΔV; in other words, the net repulsive force between the colloids have increased $\Delta\bar{\sigma}$. If $\Delta\bar{\sigma}$ is removed, the colloids will move apart a distance of Δ2d, since force and spacing are uniquely related.

The compression and expansion of colloids which has just been explained in terms of the Gouy-Chapman double layer theory can also be considered in terms of osmotic pressure. In Fig. 5d is shown the classic demonstration of osmotic pressure. On the left of a membrane, which is permeable to water molecules but impermeable to sugar molecules, is a sugar solution, and on the right is pure water. The column of fluid in the sugar solution is higher than that in the water, the difference being the osmotic pressure. One way to look at the phenomenon is to consider the water pressure on both sides of the membrane as equal. To the water pressure on the left must be added the pressure of the sugar molecules which are held back by the membrane. Thus the total pressure on the left, made up of water pressure plus sugar "pressure", is greater than that on the right by an amount equal to the sugar pressure, i.e., osmotic pressure.

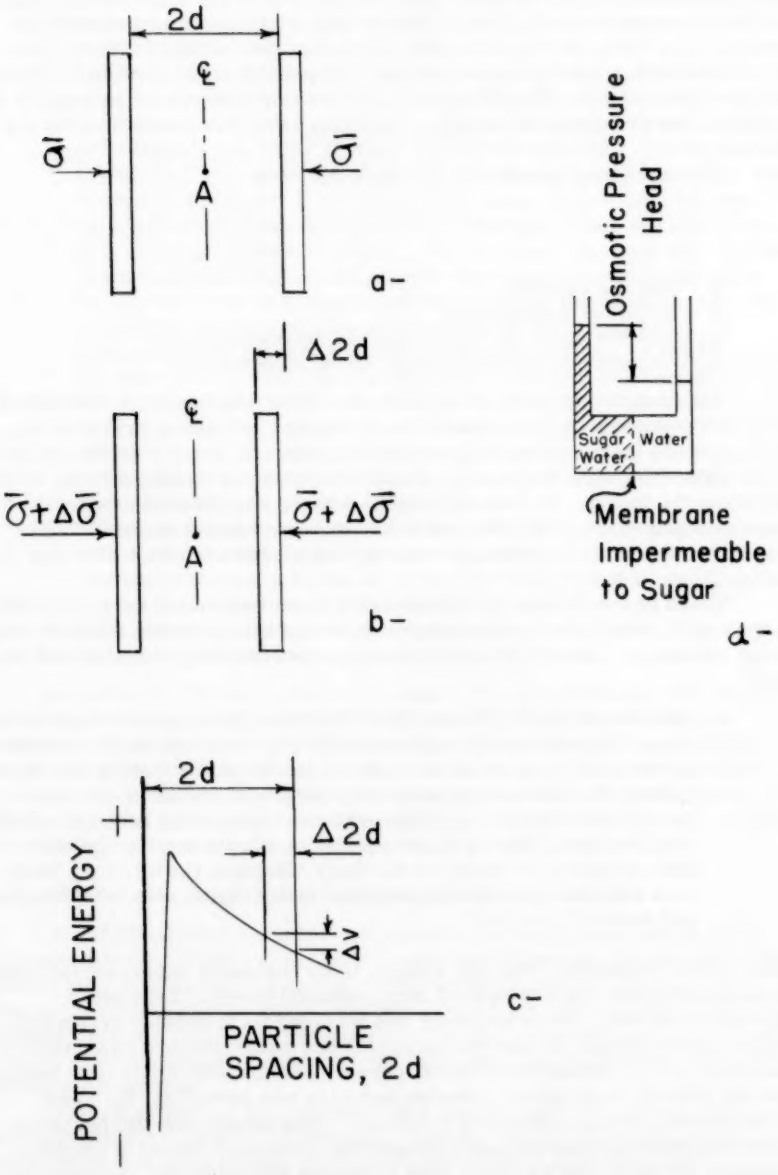


Fig. 5 — Interparticle Spacing as Function of Forces

The two particles shown in Fig. 5 present a similar situation to the model in Fig. 5d. The electrical charges on the colloids keep the cations between the particles from moving freely to the bulk of the suspension to equalize the concentrations of cations. The colloid charges act as a membrane, permeable to water and impermeable to double layer cations. There must exist a difference in total pressure between the point A and a point in the fluid far from the colloids. This difference, i.e., osmotic pressure, is equal to the intergranular pressure, $\bar{\sigma}$, plus the electrical attractive pressure (van der Waals).

The osmotic pressure, p, can be found from,

$$p = RT(n_d - n_o)$$

where,

R = the gas constant
T = absolute temperature
n_d = concentration of cations midway between colloids
n_o = concentration of cations in bulk of suspension.

The osmotic pressure is actually the electrical repulsion between colloids; this repulsion is a function of the difference in cation concentration, $n_d - n_o$. Since these concentrations must be obtained from some theory, such as the Gouy-Chapman, the osmotic pressure concept is simply one way of interpreting the theory. Its value lies in its helping one visualize expansion and compression in clays. The electrical double layer concept is more fundamental than the osmotic pressure concept, and appears to be a better way to explain Component 5.

There is convincing experimental evidence supporting the theory that Component 5 contributes to clay compression and is essentially the sole cause of clay expansion. Four of these confirming experimental evidences will be cited.

1. Bolt and Miller (1955) and Bolt (1956) ran compression tests on clays (illite and montmorillonite) with various electrolyte concentrations in the pore fluids. After a few cycles of loading and unloading to minimize particle rearrangement in later runs, they found a reversible void ratio-pressure curve, i.e., rebound equalled compression. The void ratio-pressure curves were completely understandable in terms of the Gouy-Chapman theory; e.g., whenever salt was added to depress the double layer, less compression and rebound resulted.*

* Later work (Warkentin, Bolt and Miller, 1957) compared experimental compression data with plots computed from colloidal theory. Excellent agreement between theory and tests was obtained from sodium-montmorillonite, after a couple of load-unload cycles had made the particles parallel. Warkentin, et al, assumed for the dispersed systems with which they worked that the electrical attraction between particles was zero, i.e., the swell pressure equalled the electrical repulsion. This assumption did not prove to be true with calcium-montmorillonite; the measured values of swelling pressure (σ) were much greater than computed repulsion (R). To assume that the electrical attraction A, equals zero in the soil systems with which the engineer usually works seems unjustified.

2. Ladd's work (1957), described in the next section of this paper, showed that the swelling of compacted clay can be explained completely by Component 5. Changing the electrolyte in the water available for imbibition, he was able to alter the measured swelling as had been theoretically predicted.
3. To check the belief that Component 5 was the cause of clay expansion, a test was conducted in which the effect of temperature was investigated. A sample of Boston blue clay was consolidated at a temperature of 20° C; the temperature was then raised to 41° C while the load was held constant; and finally the temperature was lowered again to 20° C under constant load. Under constant load, the clay compressed with the temperature increase and expanded with the temperature decrease. These volume changes are exactly those expected from the effect of temperature on the double layer—i.e., an increase in temperature will depress the double layer, and a decrease in temperature will expand the double layer.
Unknown to the author at the time this simple test was run was the fact that Gray (1936) had run standard consolidation tests at different temperatures (10° C and 22° C). Gray did not conduct his tests in such a way that actual volume changes could be determined for constant pressure but varying temperature. His data do, however, show clearly that the void ratio-log pressure curve can be moved up or down by altering the temperature.
4. Many experimenters (e.g., Cornell, 1951; Norrish, 1954) have shown that the basal spacing in a soil particle of expansive mineral (montmorillonite, nontronite, halloysite ($4H_2O$), vermiculite, etc.) varies directly with the water pick-up. As a matter of fact, standard laboratory techniques of mineral identification employ the change of platelet thickness with polar fluids.

In summary, the compression of saturated clay resulting from an increase in pressure is due primarily to the reorientation of particles and to a decrease in the size of micells. The rebound or expansion on load release (also swelling upon water imbibition) is due almost entirely to an increase in the size of the micelle. This theory leads us to predict that any change in the system variables listed in Section I that changes the double layer can change the compression-expansion characteristics of the clay.

B. Effect of One Dimensional Compression on Structure

Fig. 6 illustrates the effects of one-dimensional compression on the structure of clays. If the effects of long periods of time on clay deposits are ignored, both natural and compacted clays can be treated in the same discussion. A sample of clay compacted dry of optimum and a natural clay would both tend toward a random orientation of particles; a sample compacted wet of optimum or a sample of natural clay which is remolded would both tend toward a parallel orientation.

The upper part of Fig. 6 shows what would happen to the particle orientations under a one-dimensional compression. The orientations in the circles show that a load tends to align the particles in a parallel array. In the case of the remolded or wet-side compacted clay in which the particles are already parallel, a load merely brings them closer together.

The lower half of Fig. 6 suggests what would happen to particle

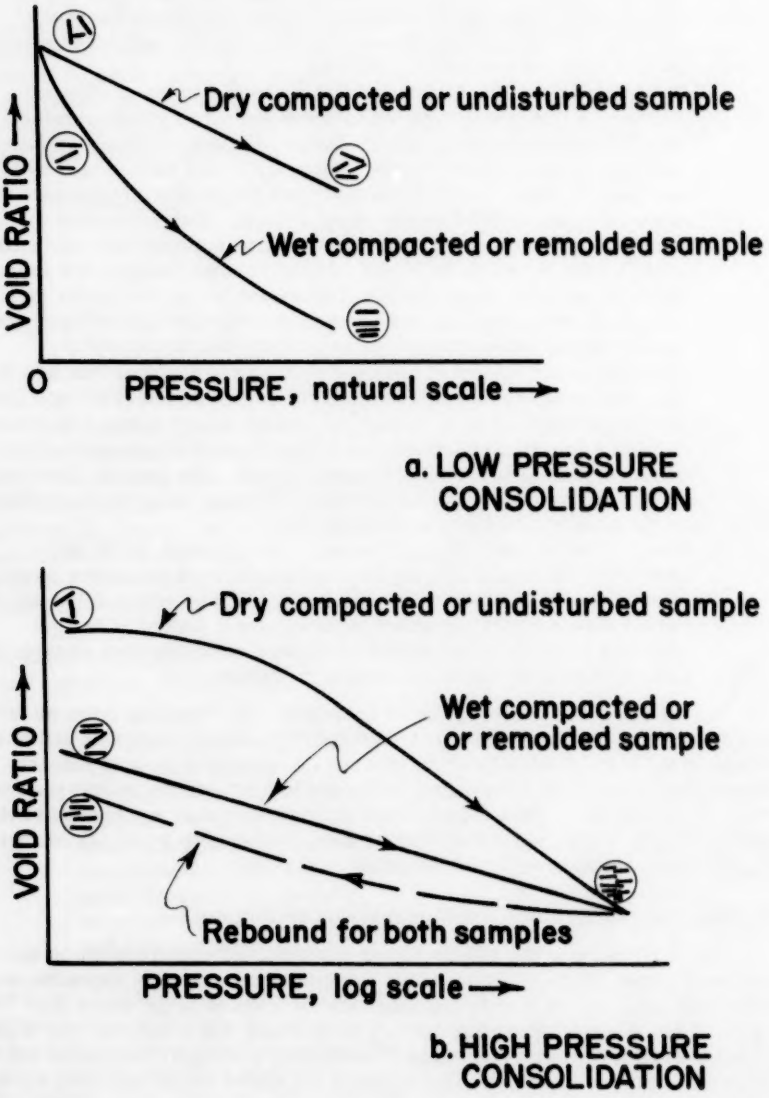


Fig. 6 — Effect of One Dimensional Compression on Structure

orientations under very high pressures. For the dry-side compacted sample the pressure orients the particles normal to the direction of compression and decreases the spacing between them. For the wet-side compacted clay in which the particles are already oriented the pressure reduces the spacing between particles. Theoretically, a large enough pressure would make the structure of the wet-side compacted sample identical to that of the dry-side compacted sample. Upon load release from this pressure the two samples would follow the same rebound. During rebound the spacing between particles increases, but only negligible changes in orientation occur.

C. Effect of Molding Water on Compressibility Characteristics of Compacted Clay

As was shown in Fig. 5, for any given pressure system there is an equilibrium volume, i.e., an equilibrium amount of water which the clay will adsorb. If a sample of clay is compacted at a moisture content less than the equilibrium water content of the clay, the clay will try to draw in more water to satisfy its double layer requirements. If an outside source of water is not available, capillary menisci will develop and tensions in the pore water will result. The pore water tension will create an intergranular pressure of equal magnitude to satisfy the double layer of the soil colloid, i.e., since there is no water to satisfy the colloid under a small intergranular pressure, the intergranular pressure increases to reduce the amount of moisture necessary for equilibrium.

When two samples at the same density, one compacted dry of optimum and one compacted wet of optimum, are compared, it is evident that the sample compacted dry of optimum must adsorb more moisture to come into equilibrium than the sample on the wet side. This well known fact is illustrated in Figs. 1, 2 and 13. (Fig. 13 from Ladd will be discussed in detail in the next section.)

In the bottom part of Fig. 13 is shown both the compaction curve as molded (by the solid line) and the moisture-density curve after soaking (by the dashed line) has been permitted. It can be seen that a sample compacted dry of optimum picks up more moisture and swells more than does a sample of the same density compacted wet of optimum. If sufficient time were allowed the dashed curve might eventually become a straight line.

The effect of molding water on swelling behavior can also be illustrated by tests in which are measured the pressures required to prevent samples from swelling when given access to water. Such tests were run on Boston blue clay. One sample was compacted dry of optimum to a density of 108.6 lb/cu.ft. and another was compacted wet of optimum with the same effort to the same density. A pressure of 1.01 Kg/cm^2 was required to prevent the dry-side sample from swelling and 0.12 Kg/cm^2 to prevent the wet-side sample from swelling.

As Fig. 6 indicates, the amount of compression which occurs under an increment of load when the pressure is small will cause a greater settlement on a wet-side compacted sample than on a dry-side compacted sample, if they are both initially saturated and at the same void ratio. On the sample compacted wet of optimum no work is required to orient the particles; extra pressure is required to do this on the flocculated structure.

The lower part of Fig. 6 shows that under higher pressures the slope of the void ratio-log pressure curve is steeper for the dry-side than for the

wet-side sample. In other words, when the pressure is large enough to cause particle reorientations, greater volume change per increment of pressure results than when no orientation changes are taking place.

Fig. 6b also shows that there is a certain pressure required to start significant changes in particle orientations. This pressure might be interpreted as the preconsolidation load on the compacted sample. This "equivalent preconsolidation pressure," however, is not necessarily caused by a pressure or its equivalent in compactive effort. The theory would indicate, for example, that the longer the dry-side compacted sample had stood under load, the higher would be the so-called preconsolidation pressure. This pressure would also depend upon the characteristics of the pore fluid. In other words, time and chemical changes can give a shape to the consolidation curve of a compacted clay and possibly a natural clay which we would normally interpret as a preconsolidation pressure. The effects on other properties of the sample may be different when the change of slope in the void ratio-pressure curve is caused by chemical factors or time rather than by pressure.

III. Shear Strength of Compacted Clay

A. Forces Between Particles

The shear strength of a soil can be thought of as made up of a granular-type strength plus a colloidal-type strength. The strength of a clean sand, for example, would be of the granular type and that of a plastic clay of the colloidal type. A soil of both colloids and granular particles would have both types of strength. The nature of granular strength, which is comparatively well-known, is outside the scope of this paper and it is not considered herein.

The first step in a study of the shear strength of two adjacent colloids is to consider the entire force system. There are four horizontal forces (if unit area is assumed, force and stress can be used interchangeably) which can act between the adjacent colloids shown in Fig. 7; these are:

1. $\bar{\sigma}$ = the externally applied intergranular stress
2. A = the electrical attraction
3. R = the electrical repulsion
4. I = the steric or geometric interaction, i.e., contact pressure.

The intergranular stress, also called the effective stress, is derived from some other source than electrical stresses originating within the particles. It is usually the most fundamental stress the soil engineer can measure or compute; under certain conditions, it is equal to the total applied normal stress minus the pore water stress.

The stress A is derived primarily from van der Waals forces. It can act over large distances, i.e., more than 100A. There can be other components in A; e.g., the electrostatic attraction occurring in non-salt flocculation.

The stress R arises primarily from the electrostatic repulsion between clay particles, which carry net negative charges. R is very sensitive to the nature of the soil water system. Changes in R bring about changes in the degree of flocculation or dispersion of colloids.

The contact pressure I, like R, is a repulsive stress; it acts only, of course, when the adjacent particles are in contact. The contact can be either between mineral surfaces or between molecules of tightly held water. The stress I is electrical in nature, but can be caused by forces originating either within the particles or outside them.

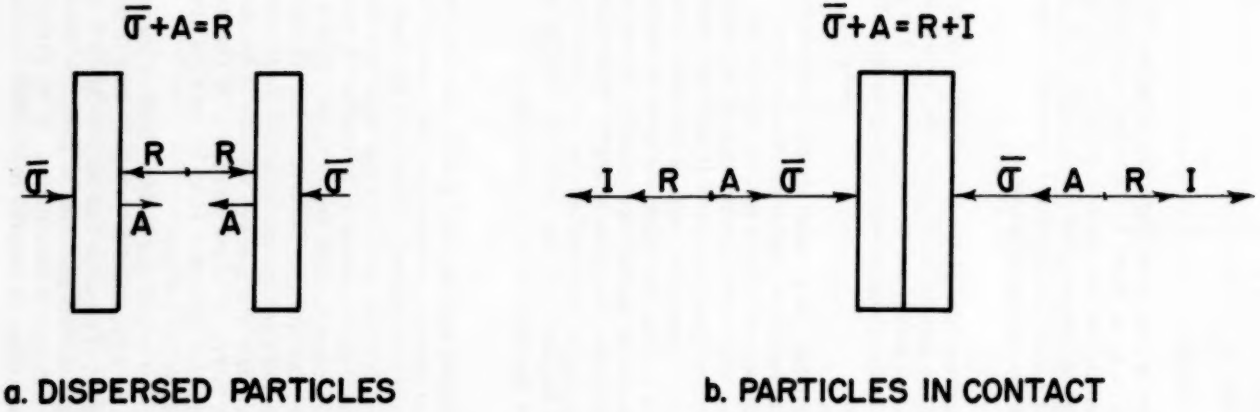


Fig. 7 — Forces Between Adjacent Particles

Equilibrium conditions require:

a. for dispersed particles, $\bar{\sigma} + A = R$
b. for particles in contact, $\bar{\sigma} + A = R + 1$.

B. Factors Influencing Shear Strength

The shear strength of a clay is completely determined by the electrical forces acting between the particles, i.e., A, R and I. The electrical forces are thus the primary cause of strength; the four factors, particle spacing, particle orientation, externally applied stresses and characteristics of soil-water system, which determine the electrical forces are secondary causes of shear strength. These five causes are discussed below:

1. Electrical Forces
   The electrical forces between clay plates are a function of particle displacement in a direction parallel to the particles, as well as a function of displacement perpendicular to the particles. The greater A and I, the greater is the shear strength; the greater R, the lower the shear strength.
2. Spacing of the Particles
   The closer two clay particles, the greater are all of the electrical forces. If two clay particles are spaced some distance apart (i.e., where I = O), the effect of a reduction in spacing on the shear strength would depend on whether the net increase in attraction is positive or negative. At the close interparticle spacings which exist in the typical natural or compacted clays, a decrease in spacing results in an increase in I. This increases the shear strength. Since increasing the externally applied intergranular pressure reduces the spacing, it increases the shear strength. This reasoning supports the well-known fact in soil engineering that, other things being equal, the denser the soil, i.e., the closer the average particle spacing, the greater the shear strength (Rutledge (1948), Leonards (1955), etc.)
3. Orientation of Particles
   For a given average particle spacing, i.e., given void ratio, the more nearly parallel the adjacent particles are, the weaker the soil. Fig. 8a and 8b show two adjacent colloids with the same average spacing Δ. The total of I plus net electrical attractive force between the adjacent particles is greater in 8b than in 8a; therefore, the shear stress required to slide the particles relative to each other is greater for the situation pictured in Fig. 8b than for that in 8a. Since the sum of I and the net attractive force between the particles varies inversely with a power function of the spacing, tilting the two particles in Fig. 8a to the position in 8b results in a greater increase of I plus net attractive force on the right side of the particles than is lost on the left side.
   The foregoing discussion of forces and strength treated only particles which are parallel or approximately parallel. This is the most common arrangement of adjacent soil colloids since it is the one which exists among dispersed particles or particles which have been flocculated with a salt type of flocculation. If two particles are perpendicular to each other as happens in non-salt flocculation

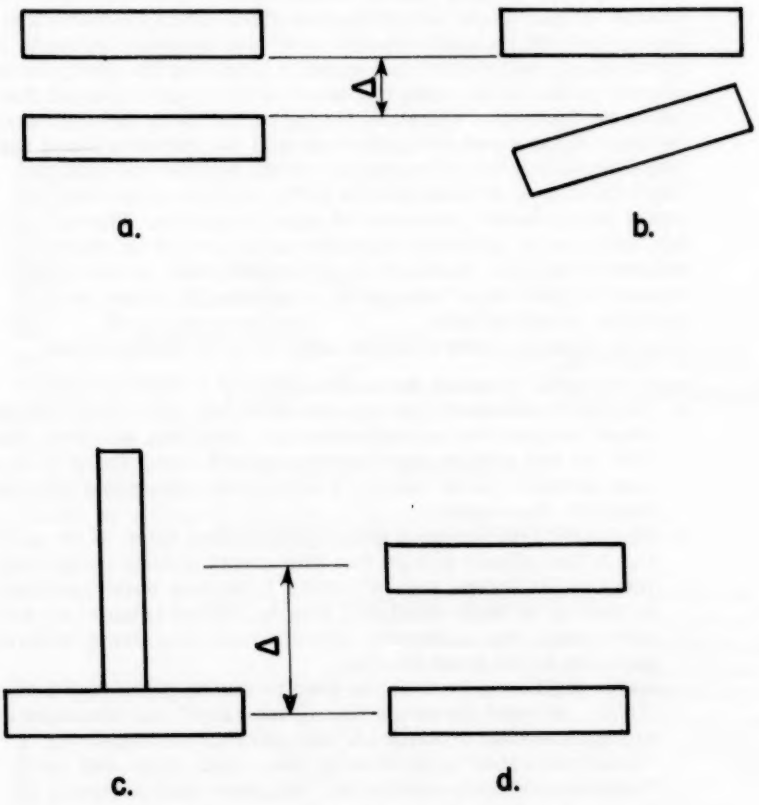


Fig. 8 — Changes in Particle Orientations

(see Fig. 8c) they resist displacement more than do the same two particles with the same average spacing but parallel as shown in Fig. 8d. With non-salt flocculation edge-to-face electrical attraction is an additional component.

When a sufficiently large shear stress is applied to the system in 8c, it rotates the particle in such a way that the orientation tends to become parallel. Experimental evidence of this tendency has already been cited. This realignment explains why the ultimate strength of a dry-side compacted sample (or an undisturbed clay sample) approaches the ultimate strength of a sample compacted wet of optimum but to the same density (or remolded sample).

The effects of particle orientation can be summarized as follows: For any given void ratio and, therefore, given average particle spacing, the more nearly parallel the particles are, the weaker the soil is.

4. Applied Stresses

A change in externally applied intergranular pressure results in a change in spacing and/or orientation which alters the electrical forces between the particles and so affects the shear strength. For particles in contact with each other, a change in the intergranular pressure affects the contact pressure between particles and thus the shear strength. The stress in the pore fluid is not a primary variable of the shear strength of the soil, but rather a factor which helps determine the intergranular stress between the particles. When the degree of saturation is 100%, negative water tensions cause intergranular pressures of equal magnitude. This is probably true also for partially saturated soils in which the degree of saturation is high. Research is needed, however, on the transmission of pore water tensions to intergranular pressures in partially saturated soils.

The stress in the pore fluid can come from several sources:

a. hydrostatic pressure due to the weight of overlying water.
b. Seepage—when particles in a saturated soil move closer to each other because they are subjected to a pressure, moisture must flow out and until an equilibrium condition exists there is an excess pressure in the water. The opposite takes place when the particles move apart.
c. Shortage of double layer water—as has been noted, if the spacing of the colloids is such that they cannot develop completely their double layers, they will swell if there is water available. If there is no water available, they will throw tension into the pore water, thus increasing intergranular pressure to hold the particles to the given spacing.

Water deficiency in soils has been treated in detail by Aitchison (1956). He used the terms "quasi-saturated" and "unsaturated" to describe soils in which the pore pressure is negative. "Quasi-saturated" soils have all their voids filled with water; "unsaturated" soils contain air. Aitchison lists a number of causes of water deficiency in soils.

5. Characteristics of the Soil-Water System

Section I discussed the effects which the variables in the soil-water system have on the electrical forces between particles. Any expansion of the double layers increases the electrical repulsive force between adjacent particles.

The attractive forces between particles are seldom altered by colloidal phenomena. Percolating waters can, however, deposit cementing agents, of which the most important are carbonates and iron oxides.

C. Cohesion and Friction

The results of soil strength tests are usually plotted as shear strength vs normal pressure (see, for example, Taylor, 1948), as shown in Fig. 9 for drained shear or for undrained shear with measured pore pressures, and in

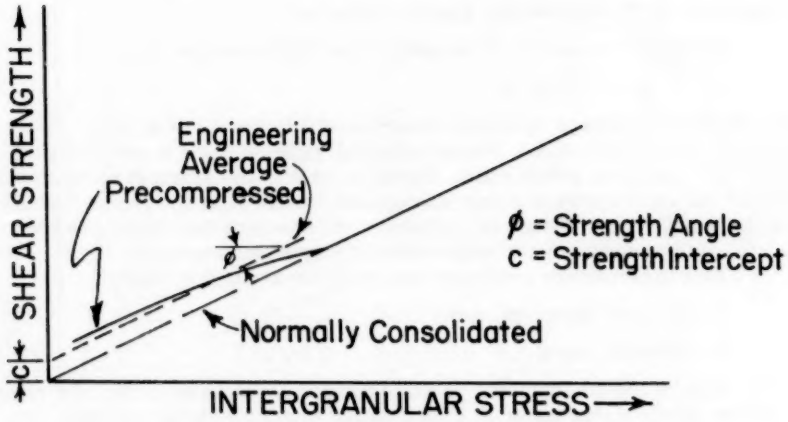


Fig. 9 — Envelopes for Drained Shear

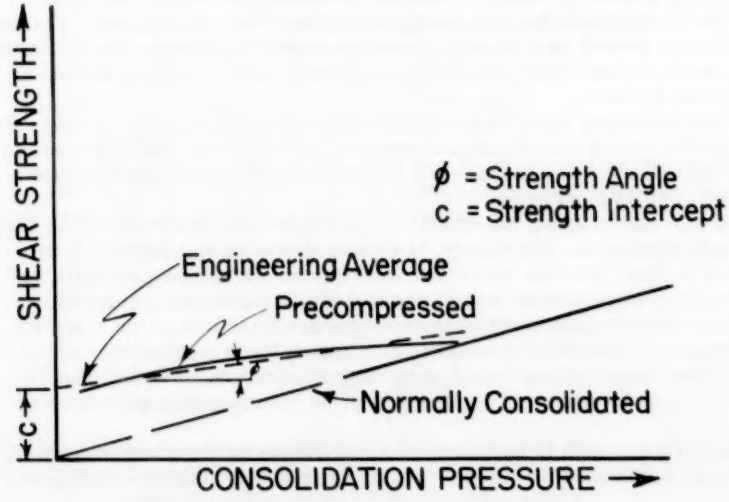


Fig. 10 — Envelopes for Undrained Shear

Fig. 10 for consolidated undrained shear without measured pore pressures.* In both figures, the following legend is used: dashed line for normally consolidated clay; solid line for precompressed clay; dotted line for straight line representation of shear strength for use in engineering analyses.

The terms "cohesion" and "friction angle" have been used in various ways to describe the strength envelopes. One common use is to name the parameters of the engineering approximation as,

Strength = cohesion + $\bar{\sigma}$ tangent of the friction angle, or

$$s = c + \bar{\sigma} \tan \phi.$$

This equation (Coulomb equation) is very useful to the soil engineer. While its use is completely sound, the selection of "cohesion" for c and "friction angle" for ϕ is most unfortunate. There is little in the strength envelopes to indicate the mechanism of shear strength development as the terms "cohesion" and "friction" imply. There is, moreover, no standard test which can identify the contributing fundamental components of clay shear strength.

This unnecessary confusion can easily be avoided by using:

c = strength intercept, and

ϕ = strength angle.

Figs. 9 and 10 show that precompression strengthens soils. As noted in earlier parts of this paper, a consolidating pressure aligns particles and reduces the spacings between them. When the confining pressure is reduced, the orientation of particles remains essentially unchanged.

The permanence of the effect of precompression on particle spacing depends on the electrical nature of the system. Particles represented by Curve 1 of the energy-distance plots in Fig. 11 gain net electrical attraction with reduced spacing. At least part of the strength increase from compression for a soil represented by Curve 1 should, therefore, remain upon load release. The same should be true for spacings less than $\underline{x}$ in Curve 2. The reduced spacings caused by precompression on particles represented by Curve 2 at distances greater than $\underline{x}$ should be temporary, since swelling would accompany load release.

This reasoning leads to an interesting prediction, namely: a clay with parallel particles and net electrical repulsion (a remolded, swelling clay tends to meet this requirement) should have the same drained strength envelope regardless of precompression.

As one can imagine, the effects of time are very important in the phenomena just described. The theory, of course, assumes equilibrium conditions and further assumes that other time effects (thixotropic strength regain, primarily) are constant for all the various strength measurements.

Since compression results in a volume decrease that is not completely recoverable upon load release, precompression has a much larger effect upon undrained shear strength than upon drained strength. The fact that the void ratio of a precompressed sample at a given intergranular pressures is

---

* Drained shear strength is taken as the shear stress on the plane of maximum stress obliquity at the strain required to develop maximum obliquity. Undrained shear strength is taken as $\frac{P_{max}}{2A} \cos \phi$ (about $.43 \frac{P_{max}}{A}$) or simply $P_{max}/2A$.

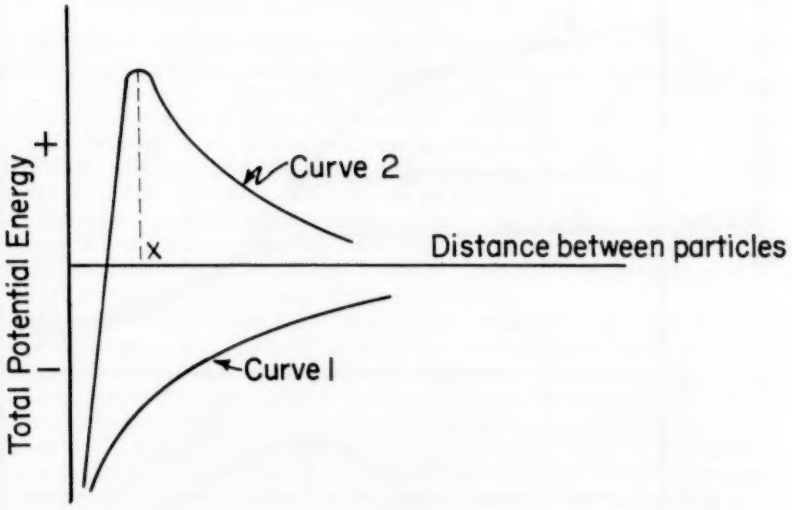


Fig. 11 — Energy-Distance Curves for Two Adjacent Colloids

lower than that of a normally consolidated sample of the same clay at the same pressure means that smaller positive pore water pressures are developed during undrained shear.

D. Effect of Compaction Conditions on Strength

1. As Molded Strength

Fig. 12* presents compaction and strength data on Boston blue clay (from Pacey, 1956). The data, typical of undrained, as-molded strengths of compacted clay, illustrate the following facts:

a. Increased compactive effort, dry of optimum, increases strength.
b. Increased compactive effort, wet of optimum, can result in a gain or loss of strength; the effect is small.
c. For the same compactive effort and same compacted density, dry-side compaction gives a higher strength than does wet-side compaction.

* Pacey's tests were made using a miniature compaction mold; the following compactive efforts were used:
Curves C - 40 lb. tamp, 3 layers of 25 tamps
Curves B - 40 lb. tamp, 5 layers of 25 tamps
Curves A - 80 lb. tamp, 3 layers of 25 tamps.

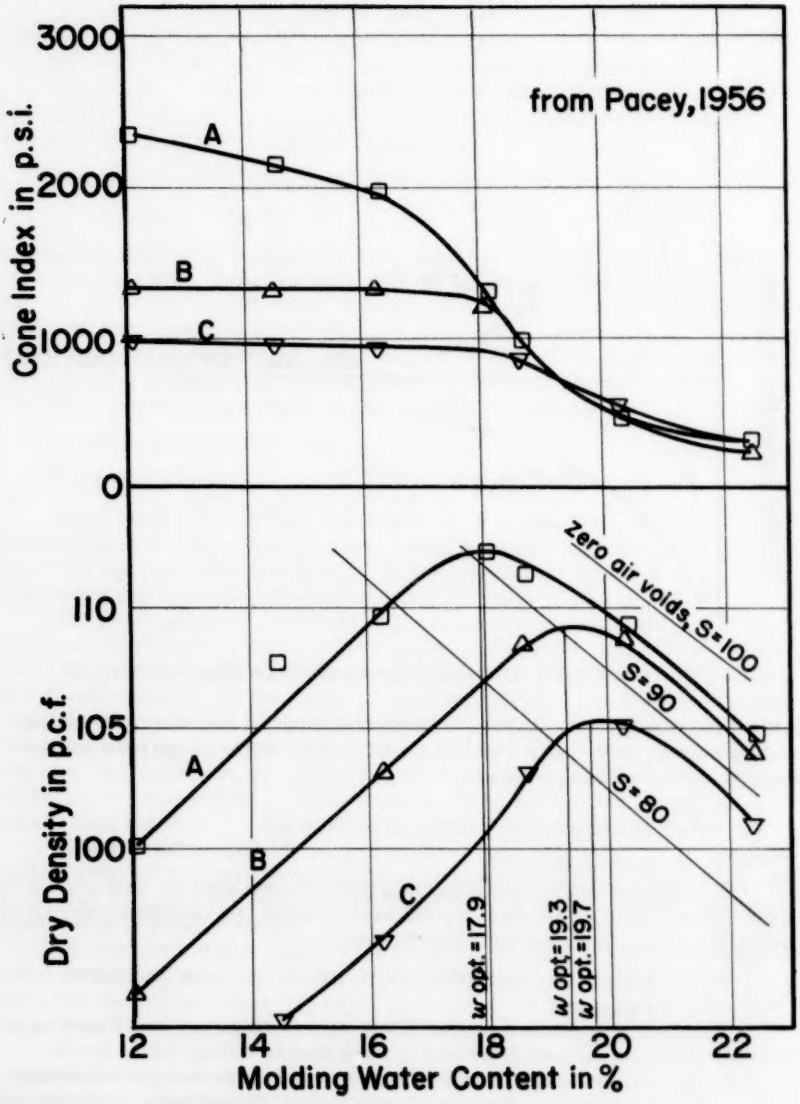


Fig. 12 — Cone Index and Dry Density vs Molding Water Content for Boston Blue Clay

These three well-known facts (see e.g., Turnbull and Foster, 1956) can be explained by the compaction theory. Comparing samples compacted at the same moisture dry of optimum but to different densities (see A and E in Fig. 9 in the first paper of this series) we see:

a. A has a more random particle orientation than E.
b. E has closer particle spacing than A.

The reduction of particle spacing in E more than offsets the improvement of orientation; going from A to E results in an increase of strength.

Going from C to D improves the orientation, which takes away strength, but makes the interparticle spacing smaller, which gives strength. Unlike the difference between A and E, the decrease in the spacing of particles (i.e., increase in density) is relatively small. The two effects are of the same magnitude and so either a net gain or a net loss of strength can occur.

Sample A is much stronger than C for three reasons: 1) the orientations at A are more random, 2) there are lower pore water pressures (higher negative pressures) at A than at C, and 3) the electrical repulsions at A are lower because of the greater water deficiency.

There is substantial direct and indirect evidence (e.g., Hilf, 1956) that clays compacted dry of optimum usually have negative pore water pressures. These negative water pressures result in higher intergranular stresses and, therefore, higher strength. Clays compacted dry of optimum do not have sufficient moisture to develop fully the double layers of the soil colloids. The colloids try to draw in the needed water; this attempt results in the development of capillary menisci which transfer water tensions into intergranular compression. Pore water tensions developed during shear (caused by a tendency toward a volume increase) also increase the undrained shear strength.

There is convincing experimental proof that dry-side compacted clays have a water deficiency and that this deficiency is due to incomplete development of the double layer. Data, taken from Ladd (1957) are presented in Fig. 13.* The bottom curves show the density-water content curve as molded and after the samples had been soaked in a mold while subjected to a surcharge of 200 lbs. per sq. ft. The sloping lines with the arrow heads show the changes of various points due to soaking. These curves show two typical characteristics of such tests, namely, that dry-side compacted samples pick up more water and swell more than wet-side samples. The higher air content in the dry-side samples helps cause them to swell more than wet-side samples.

The upper two sets of curves in Fig. 13 present strength and water pick-up data for the compacted samples. The b curves are the results of a series of tests in which samples compacted by the standard procedure with distilled water as the pore fluid were soaked in a solution of 10 N calcium chloride. The two sets of curves show:

1. Samples compacted dry of optimum have a much greater as-molded

* The curves in Fig. 13 are based on many tests; the many experimental points were not plotted in Fig. 13 in order that the trends might be more clearly seen. Curve c is for as-molded; Curve a, after soaking in distilled water; and Curve b, after soaking in 10 $\bar{N}$ $CaCl_2$.

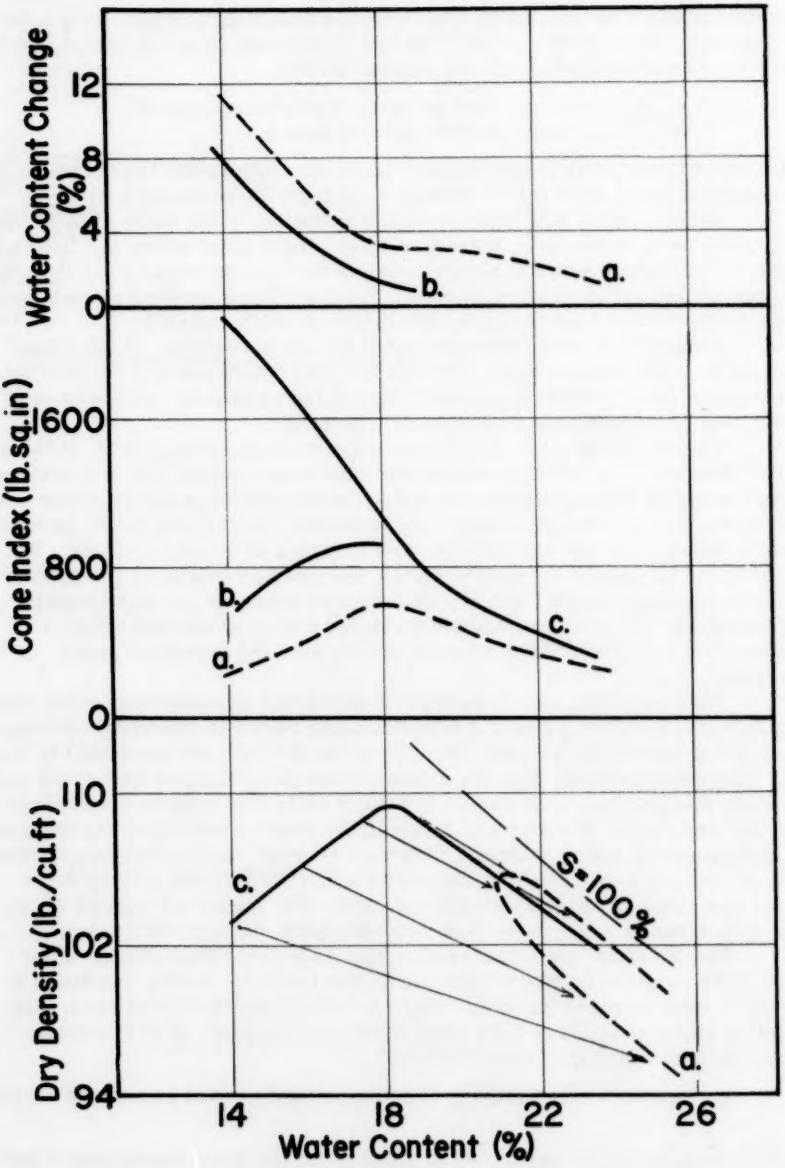


Fig. 13 — Compaction-Soaking-Strength Data for Vicksburg Buckshot Clay

undrained strength than those compacted wet of optimum.*

2. The soaked samples imbibed much less moisture when soaked in salt water than when soaked in distilled water. The samples soaked in salt water retained much more strength than those soaked in distilled water.

The lower water pick-up and strength loss from salt water soaking in comparison to those from fresh water soaking follows from our theory. The increase in pore water electrolyte depresses the double layers of the colloids, i.e., reduces the water needed for double layer development. Curtailing double layer expansion curtails strength loss.

2. Strength of Saturated Samples

If compacted samples having the same density, one dry-side (A in Fig. 9 in the first paper) and one wet-side (E in Fig. 9 in the first paper) are saturated while the volume is held constant, the dry-side sample should be stronger than the wet-wide sample, but the strength difference should be reduced by the saturation. After saturation at constant volume:

a. The water contents and electrolyte contents in the pore fluids of the two samples should be equal and, therefore, electrical repulsions should be more nearly equal.
b. The water contents of the two samples being the same, the double layer water deficiencies should be more nearly equal, which would make pore water tensions prior to shear more nearly equal.

The orientation at A would still be more random than at C, and the strength at A should, therefore, still be greater than at C. Sample A, being less compressible than C, will develop smaller positive pore water pressures during undrained shear.

Fig. 14 (from Pacey, 1956) shows that samples compacted dry of optimum and then soaked at approximately constant volume have higher shear strengths than samples compacted wet of optimum. Pacey's data were obtained from a cone penetration test. If free swelling, or even partial swelling, is permitted, the wet compacted clay may have higher undrained strength (see Fig. 13, ASCE (1950), and Seed and Monismith, (1954).

In an attempt to get more refined data, Huning, (1957) ran triaxial tests on compacted samples which were saturated and then consolidated prior to shearing. Fig. 15, presenting shear strength as a function of molding water content, shows the same trends as Pacey's work, namely: samples compacted dry of optimum and then saturated possess more strength than those compacted wet and then saturated.

Huning's strength data, plotted in Fig. 16, also show that for a given water content at failure, higher strength results from dry-side compaction. The data in Figs. 15 and 16 were obtained from undrained triaxial

* The strengths of soaked samples (dashed curve in the middle of Fig. 13) shows a slightly higher undrained strength for wet-side compaction. This is caused by the greater swelling of dry-side samples. At equal densities after soaking, the dry-side clay is stronger as expected. Under the very low confining surcharge of 200 lb/sq. ft., the excessive swelling of dry-side samples resulted in greater undrained shear for the wet-side samples.

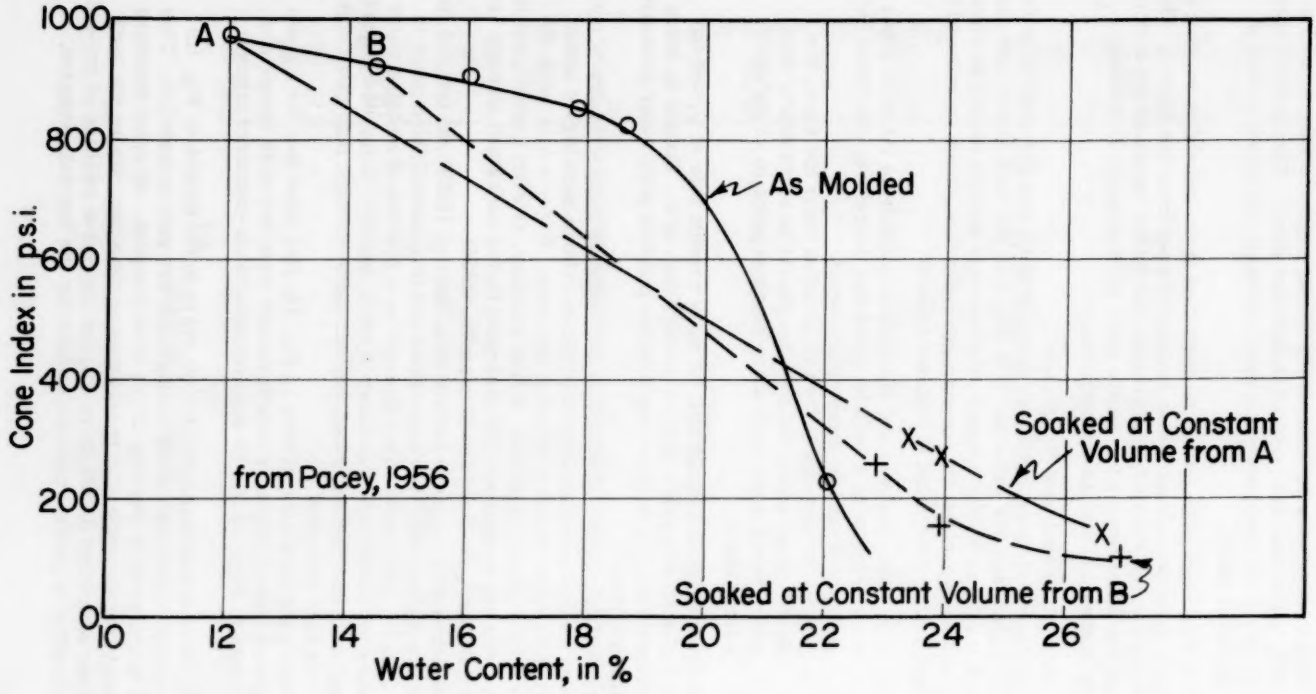


Fig. 14 — Strength vs Water Content for Compacted Boston Blue Clay

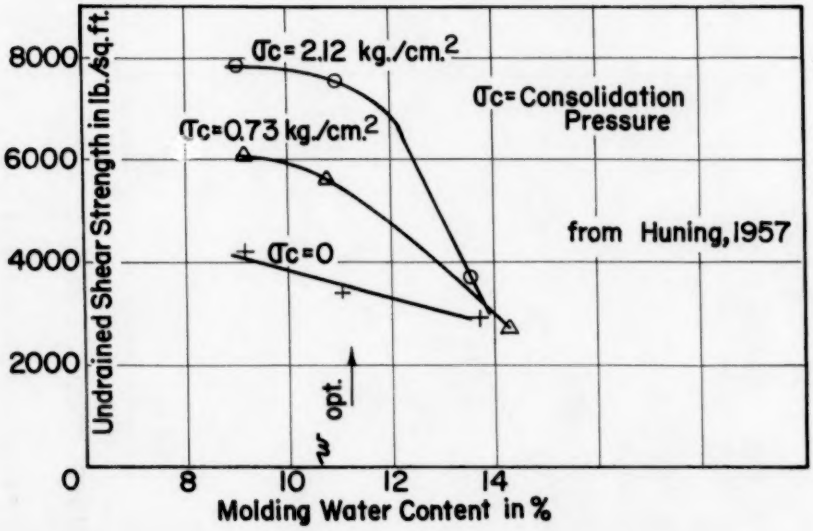


Fig. 15 — Strength vs Molding Water for Compacted La Salina Clay

tests in which pore water pressures were measured. The measured pore pressures, all tensions, shown in Fig. 17, indicate that the strength-intergranular stress plots will not differ very much for the various compaction water contents since higher pore water tensions at failure develop in samples compacted dry of optimum (except for moistures far below optimum). Fig. 18 confirms this by showing almost no variation in drained strength with molding water content or compacted density.

Huning's test procedures were such that the data do not give maximum help in the examination of the structure theory. During saturation, volume changes were not prevented. The saturation process (during which very high gradients existed) and the consolidation process subjected the samples to severe stresses, unlike any to be expected in the field. These stresses changed the soil structure. Fig. 18, as mentioned above, suggests that neither molding water content nor density had any effect on the drained strength envelope. Actually Huning's test techniques destroyed differences in soil structure which had developed during compaction.* For example, the densest samples swelled the most, thus wiping out the benefits of high density. Further tests are needed to show clearly the effect of compaction on drained shear strength.

* Leonards (1955) noted that it was not possible to saturate a compacted clay without disturbing the structure.

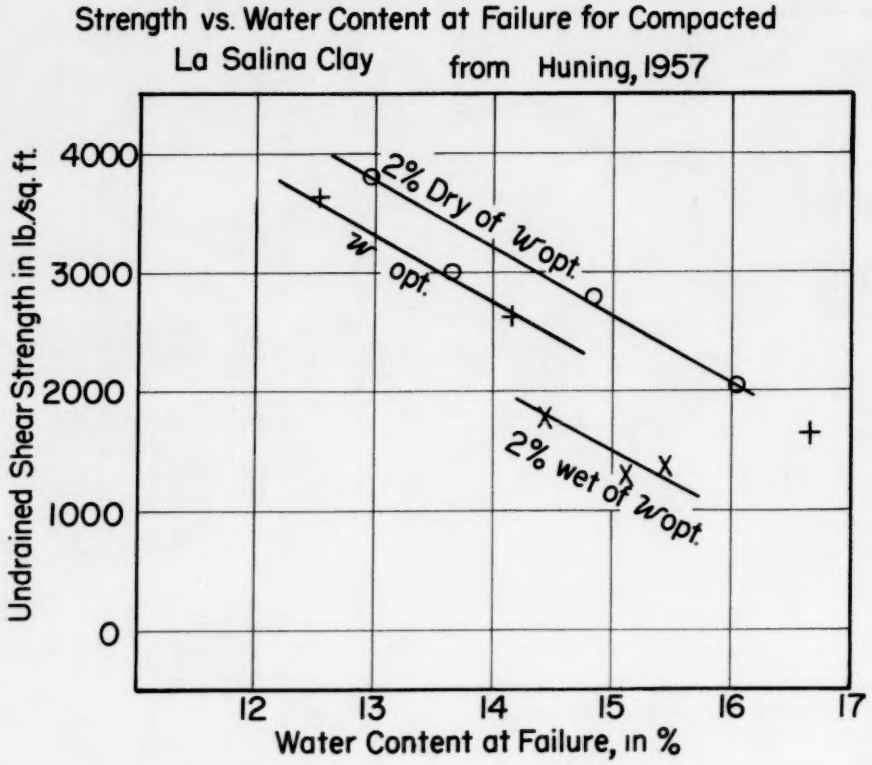


Fig. 16 — Strength vs Water Content at Failure for Compacted La Salina Clay

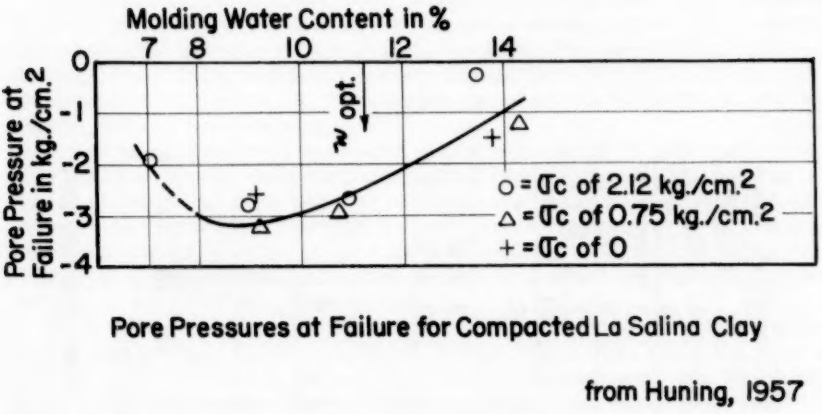


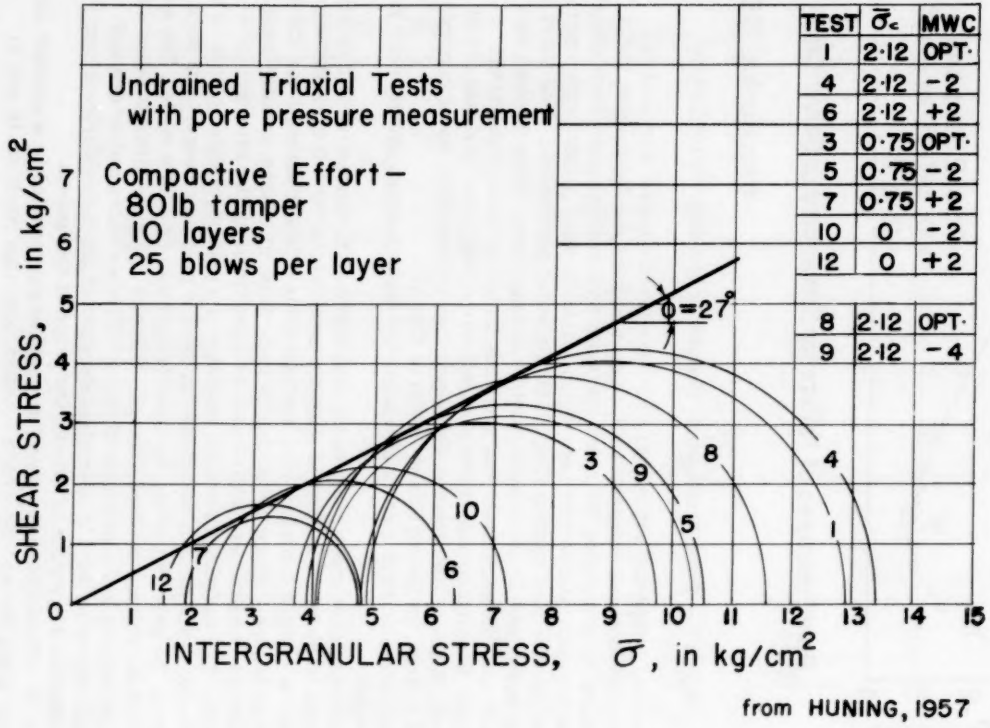
Fig. 17 — Pore Pressure at Failure vs Molding Water Content

Samples compacted dry of optimum have much steeper stress-strain curves than samples compacted wet (e.g., see Seed and Monismith, 1954). In fact, samples compacted far below optimum tend to be brittle. Such samples can have a meta-stable structure which collapses upon contact with water, just as the air-mica system (Fig. 4) did on water permeation. The water permits a more orderly and denser particle arrangement. This throws compression into the pore water and produces a marked loss in undrained strength. In samples with water contents more than a few per cent dry of optimum, there is less tension developed than in those with water contents dry of, but nearer, optimum; this is shown in Fig. 17.

The curves of axial load vs strain for wet-side samples usually show no peak value.

In recent years much controversy has raged as to whether ϕ or c or either one ought to be taken as zero in stability analysis. A "ϕ = O analysis" really amounts to using a constant, i.e., average value of shear strength for the entire trial failure surface being considered. This, of course, greatly simplifies the stability computations and is just as accurate as is the selection of strength. The undrained strength envelope of a preconsolidated clay or a compacted clay tends to have little slope in the low pressure range. For such a condition, the selection of an average strength can be easily made. The selection of an average drained strength, however, is more difficult since the envelope usually has a pronounced slope. Whether undrained or drained strength should be used in a problem depends on the particular problem; a discussion of this choice is outside the scope of this paper.

In a "c = O analysis" the shear strength is represented by a straight line envelope going through the origin. Essentially, this condition is met in normally consolidated clays. The c = O method of analysis, as practiced, disregards the effects of precompression or compaction in giving strength greater than that indicated by the straight envelope through the origin. Here again, the reasonableness of this severe and conservative assumption depends on the particular soil and problem at hand. More understanding on the permanence of the effects of precompression and compaction are certainly needed.


Undrained Triaxial Tests
with pore pressure measurement
Compactive Effort—
80 lb tamper
10 layers
25 blows per layer
TEST | $\bar{\sigma}_c$ | MWC
1 | 2·12 | OPT·
4 | 2·12 | −2
6 | 2·12 | +2
3 | 0·75 | OPT·
5 | 0·75 | −2
7 | 0·75 | +2
10 | 0 | −2
12 | 0 | +2
8 | 2·12 | OPT·
9 | 2·12 | −4
ϕ=27°
SHEAR STRESS, in kg/cm²
INTERGRANULAR STRESS, $\bar{\sigma}$, in kg/cm²
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
1 2 3 4 5 6 7

from HUNING, 1957

Fig. 18 — Strength Envelope for La Salina Clay

The proper selection of the shear strength (or components in the Coulomb equation for strength) depends on the soil and problem at hand. This selection can be difficult and involves certain assumptions. The use of a horizontal envelope ("$\phi = 0$") or straight envelope through the origin ("$c = 0$") can be correct and very convenient. The blanket use of either, however, is poor engineering and can be disastrous.

In summary, the theory expounded in this paper predicts and experimental data confirm that dry-side compaction gives clay a structure that results in higher strength than does wet-side compaction. Water imbibition at constant volume destroys some of the differences in structure; most of them remain. Since wet-side compaction gives a more compressible structure, higher pore pressures are developed during shear. This pore pressure build-up accounts for part of the lower undrained strength.

Even more of the differences in structure are destroyed if swelling is permitted during water imbibition. Shear strains, which align particles, also affect the structure built in during compaction. Disturbances from consolidation water imbibition, swelling, and large shear strains can result in a "true" shear strength envelope which has no relation to compaction conditions.

Summary and Conclusions

This series of papers on structure presents a mechanistic picture of the process of clay compaction. The effects of compactive effort and molding water content on the soil structure are described. The relationships between structure and engineering behavior are discussed and illustrated with experimental data. Colloidal theory is used to present the effects of environment on soil structure and on changes in soil structure. There are similarities between the structures of clay compacted dry of optimum and of undisturbed clay and between those of clay compacted wet of optimum and of remolded clay.

Table 1 summarizes the comparison between a sample compacted to a given density dry of optimum and a sample of the same clay compacted to the same density at a water content wet of optimum. The answer to: "which is better, dry-side or wet-side compaction?" depends on the soil and problem involved. For example: 1) A low clay dam on a compressible foundation should be compacted wet. 2) A high highway clay fill to be built very rapidly on a good foundation should be compacted dry.

This paper has, of necessity, neglected factors which could be important, namely:

1. The effects of silt and sand particles.
2. The effects of chemical reactions in the soil which result in new products, especially cementing agents.
3. Certain electrical forces which can act between particles at close spacing.

These factors would not have any fundamental effects on the theories presented, but would influence the magnitude of the trends postulated.

The experimental data presented in this paper are typical of the results obtained from many tests run during the past few years in the M. I. T. Soils Laboratory. Limitations of space prevented inclusion of more of these data and also prevented an examination, in the light of the theoretical concepts

given by this paper, of some of the published data by soil engineers from other organizations.

These papers are not intended to be the last word, but rather an over-all guide to the understanding of the behavior of fine-grained soils, especially compacted clays. Several topics presented in this paper need critical theoretical and experimental study; among them are:

1. Soil Moisture—nature of the moisture near the clay particles, especially the effect of time on its behavior.
2. Pore Pressures—a study of pore pressures in clay, especially partially saturated clays, to determine such things as the minimum degree of saturation at which pore pressures can be measured and the range of validity of the expression, Total = intergranular + neutral pressure.
3. Strength—a very detailed mechanistic picture of shear strength of clay and of clay containing silt and sand particles. An experimental study of the variation of strength with changes in the factors discussed in Section IX B.
4. Environmental Changes—an investigation of the effect of changes in the characteristics of the soil-water system (temperature, etc.) on soil structure and the effect of changes in soil structure.
5. Soil Structure—further correlations of structure with behavior; development of simple test to determine structure.
6. Type of Compaction—the effects of compaction type on structure and behavior should be studied first in the laboratory and then in the field. The development of more convenient and more representative laboratory compaction tests could result. Better field compaction techniques would result.

Much of the compaction of clay today is a fairly crude and yet expensive operation. The designer of clay embankments is faced with many unknowns. The development of knowledge about compacted clay can result in more economical designs and cheaper unit field compaction. An understanding of the fundamentals of compacted clay can greatly increase the likelihood of finding chemical additives which can be widely and economically used to improve the properties of clay.

ACKNOWLEDGMENTS

The author is happy to acknowledge the help of many in obtaining and in presenting the information in this series of papers. The results of the author's research students (J. G. Pacey, I. Tiles, R. C. Huning, H. W. Olsen, G. J. Haggarty, and Dr. James K. Mitchell) have been used.

Staff members in the M. I. T. Soil Engineering Division (J. M. de Souza, R. Harkness, O. H. Gilbert, Jr., T. J. Lambie, Z. C. Moh, P. S. Parsonson, and J. E. Roberts) have contributed to this paper by examining critically the concepts presented and making suggestions for their improvement. C. C. Ladd, Instructor in the Soils Division, was particularly helpful in this respect.

Professor A. S. Michaels of the M. I. T. Chemical Engineering Department and Dr. R. Torrence Martin, Research Associate in the Soil Engineering Division, have spent many hours trying to explain the mysteries of colloid chemistry and crystal chemistry to the author.

The long discussions the author has had with his colleague, Prof. Robert V. Whitman, have had a significant effect on his thinking. Professor Whitman's

comments and suggestions have considerably improved the theories presented in this paper.

All of the staff of the Soil Engineering Division studied drafts of this paper and made suggestions. The author, however, willingly accepts, as he must, the responsibility for the material presented.

TABLE 1

Comparison Dry-of-Optimum with Wet-of-Optimum Compaction

Property	Comparison
1. Structure	
A. Particle Arrangement	Dry-Side more random.
B. Water Deficiency	Dry-Side more deficiency ∴ imbibe more water, swell more, have lower pore pressure.
C. Permanence	Dry-Side structure sensitive to change.
2. Permeability	
A. Magnitude	Dry-Side more permeable.
B. Permanence	Dry-Side permeability reduced much more by permeation.
3. Compressibility	
A. Magnitude	Wet-Side more compressible in low pressure range, dry-side in high pressure range.
B. Rate	Dry-Side consolidates more rapidly.
C. Rebound	Wet-Side rebound per compression greater.
4. Strength	
A. As Molded	
a. Undrained	Dry-Side much higher.
b. Drained	Dry-Side somewhat higher.
B. After Saturation	
a. Undrained	Dry-Side somewhat higher if swelling prevented; wet-side can be higher if swelling permitted.
b. Drained	Dry-Side about the same or slightly greater.
C. Pore Water Pressure at Failure	Wet-Side higher
D. Stress-Strain Modulus	Dry-Side much greater.
E. Sensitivity	Dry-Side more apt to be sensitive.

REFERENCES

1. ASCE, "Development of CBR Flexible Pavement Design Method for Airfields," A Symposium, ASCE Trans. Vol. 115, p. 453, 1950.
2. Bolt, G. H., "Physical-Chemical Analysis of the Compressibility of Pure Clay," Geo. Technique, Vol. 6, No. 2, pp. 86-93, June, 1956.
3. Bolt, G. H., and Miller, R. D., "Compression Studies of Illite Suspensions," Soil Science Proc. 19:285-288, 1955.
4. Cornell University, "Final Report, Soil Solidification Research, Vol. II—Fundamental Properties, Clay-Water Systems," Ithaca, New York, 1951.
5. Gray, Hamilton, "Progress Report on Research on the Consolidation of Fine-Grained Soils," Proc. International Conference on Soil Mechanics and Foundation Engineering, Harvard Univ., 1936.
6. Hilf, J. W., "An Investigation of Pore Water in Compacted Cohesive Soils," T. M. 654, U. S. Dept. of the Interior, Bureau of Reclamation, Denver, Colorado, October, 1956.
7. Huning, R. C., "The Effect of Structure on the Strength of Compacted Soil," S. M. Thesis, M. I. T., 1957.
8. Hvorslev, Juul, "The Shearing Resistance of Remolded Cohesive Soils," Proc. Soils and Foundation Conference, U. S. Engr. Dept., Boston, Mass., June, 1938.
9. Ladd, C. C., "Swelling of Compacted Clay," S. M. Thesis, M. I. T., 1957.
10. Lambe, T. William, "Structure of Inorganic Soil," ASCE Separate 315, October, 1953.
11. Lambe, T. William, "The Permeability of Compacted Fine Grained Soils," ASTM, 1955.
12. Leonards, G. A., "Strength Characteristics of Compacted Clay," ASCE Trans., 1955.
13. Michaels, Alan S., and Lin, C. S., "The Permeability of Kaolinite," Industrial Eng. Chem., Vol. 46, p. 1239, 1954.
14. Michaels, Alan S., and Lin, C. S., "Effects of Counter Electro-Osmosis and Sodium Ion Exchange on the Permeability of Kaolinite," Industry and Engineering Chemistry, 1955.
15. Mitchell, James K., "The Importance of Structure to the Engineering Behavior of Clay," Sc.D. Thesis, M. I. T., 1956.
16. Norrish, K., "The Swelling of Montmorillonite," Faraday Society Discussion, No. 18, 1954.
17. Pacey, John G., Jr., "The Structure of Compacted Soils," S.M. Thesis, M. I. T., 1956.
18. Rutledge, P. C., "Strength of Natural Clays," presented at the October 1958 Meeting of A.S.C.E.
19. Seed, H. B. and Monismith, Carl L., "Some Relationships Between Density and Stability of Subgrade Soils," Highway Research Board, 1954.

20. Taylor, D. W., Fundamentals of Soil Mechanics, John Wiley, 1948.

21. Trollope, D. H., "The Basic Law of Shear Strength," Proc. 1st Australian-New Zealand Conf. on Soil Mech. and Fdn. Engr., 1952.

22. Turnbull, W. K. and Foster, C. R., "Stabilization of Materials by Compaction," ASCE Separate 934, April, 1956.

23. Warkentin, B. P., Bolt, G. H., Miller, R. D., "Swelling Pressure of Montmorillonite," Soil Science Proc., Vol. 21, No. 5, Sept.-Oct., 1957.

Paper 1657 May, 1958 SM 2

Journal of the

SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1657)

Page

Earthquake Resistance of Rockfill Dams, by Ray W. Clough and David Pirtz. (Proc. Paper 941, April, 1956. Prior discussion: 1094. Discussion closed.)
by Ray W. Clough and David Pirtz (closure) 1657-3

Frictional Resistance of Steel H-Piling in Clay, by Eben Vey. (Proc. Paper 1160, January, 1957. Prior discussion: 1228, 1314, 1430. Discussion closed.)
by Eben Vey (closure) 1657-7

Field Experience with Chemical Grouting, by Milos Polivka, Leslie P. Witte and John P. Gnaedinger. (Proc. Paper 1204, April, 1957. Prior discussion: 1430. Discussion closed.)
by Milos Polivka, Leslie P. Witte, and John P. Gnaedinger (closure) 1657-9

Compacting Earth Dams with Heavy Tamping Rollers, by Jack W. Hilf. (Proc. Paper 1205, April, 1957. Prior discussion: 1430. Discussion closed.)
by Jack W. Hilf (closure) 1657-11

Dewatering Miami's Biscayne Aquifer, by Byron J. Prugh. (Proc. Paper 1299, July, 1957. Prior discussion: 1430. Discussion closed.)
by Byron J. Prugh . . . . . - - -*

A Review of the Theories for Sand Drains, by F. E. Richart, Jr. (Proc. Paper 1301, July, 1957. Prior discussion: 1559. Discussion closed.)
by S. J. Johnson 1657-15
by F. E. Richart, Jr. (closure) 1657-16

(Over)

Note: Paper 1657 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, SM 2, May, 1958.

* There will be no closure.

Page

Thixotropic Characteristics of Compacted Clays, by H. B. Seed and C. K. Chan. (Proc. Paper 1427, November, 1957. Prior discussion: 1559. Discussion closed.)
by D. P. Krynine 1657-21

Laterite Soils and their Engineering Characteristics, by K. S. Bawa. (Proc. Paper 1428, November, 1957. Prior discussion: 1559. Discussion closed.)
by Jacques de Medina 1657-33
by F. L. D. Wooltorton 1657-35
by S. F. Gizienski. 1657-36

Geologic Investigations of Dam Sites by the SCS, by Gunnar M. Brune. (Proc. Paper 1429, November, 1957. Prior discussion: none. Discussion closed.)
by John A. Trantina 1657-41

Garrison Dam—Evaluation of Results from Tunnel Tests Section, by K. S. Lane. (Proc. Paper 1439, November, 1957. Prior discussion: none. Discussion closed.)
by A. A. Eremin 1657-43

A Method to Describe Soil Temperature Variation, by E. B. Penrod, W. W. Walton and D. V. Terrell. (Proc. Paper 1537, February, 1958. Prior discussion: none. Discussion open until July 1, 1958.)
by D. C. Pearce. 1657-45

Cement and Clay Grouting of Foundations: Grouting with Clay-Cement Grouts, by Stanley J. Johnson. (Proc. Paper 1545, February, 1958. Prior discussion: none. Discussion open until July 1, 1958.)
by Robert W. Cunny 1657-47

EARTHQUAKE RESISTANCE OF ROCKFILL DAMS[a]

Closure by Ray W. Clough and David Pirtz

RAY W. CLOUGH,[1] A.M. ASCE and DAVID PIRTZ,[2] A.M. ASCE.—The writers appreciate the interest in their paper which has been exhibited in the discussions, and welcome this opportunity to amplify their original presentation with respect to the points raised by the discussers.

Mr. Spielman has recognized correctly the lack of similitude in the model with respect to the crushing strength of the rock. Ideally, the ratio of model to prototype rock strength should have been the same as the ratios of modulus of rigidity and shear strengths, which would have required a crushing strength in the model rock in the neighborhood of 50 to 100 psi (as stated by Mr. Spielman). The writers were aware of this discrepancy at the time the model was designed, but disregarded it as relatively unimportant to the primary action which was being studied. It is true that local crushing of the rock could have had a noticeable effect on the settlement of the rock structure, but contrary to Mr. Spielman's statement, the magnitude of settlement was not the principal factor under investigation. The primary objective was to learn whether the structure would exhibit tendencies toward shear failure through the clay core, and local crushing should have little effect on this action. As to the actual magnitude of settlement, this would depend upon many factors more important than local crushing including the intensity and duration of the earthquake, and the degree of compaction of the rock obtained during its original placing. Consequently, only the relative magnitudes of settlement observed with different models have any significance in these tests, and the lack of similitude with respect to crushing strength would have had no effect on relative settlements.

Regarding the question raised by Mr. Spielman on the cohesion in the wet rock structure due to surface tension effects, for the size of rock particles used in the investigation (between 3/8" and No. 30 Sieve) it is believed the magnitude of these cohesive forces is completely negligible.

Mr. Ambraseys has raised several interesting points with regard to the action of the model materials during an earthquake. The principal point of contention seems to be that the increase of soil strength demonstrated in dynamic tests would not be developed during earthquakes because of the alternating nature of the earthquake motion as compared with the one-directional dynamic tests, and Mr. Ambraseys has referred to observed damage during earthquakes as well as laboratory experiments in support of this contention.

---

a. Proc. Paper 941, April, 1956, by Ray W. Clough and David Pirtz.

1. Assoc. Prof. of Civ. Eng., Univ. of California, Berkeley, Calif.
2. Assoc. Prof. of Civ. Eng., Univ. of California, Berkeley, Calif.

However, the writers believe that the available evidence supports their original claims. It must be recognized first of all that the structure under consideration does not consist of natural soils, but of carefully placed rock with a highly compacted clay core. For this reason, observations of damage due to failure of natural soils during earthquakes would appear to have little bearing on the expected response of a rock-fill dam. Similarly, the results of laboratory tests on sandy soils subjected to vertical vibrations [12] do not appear to be pertinent to the present investigations.

With regard to the rock structure of the dam both in the model and the prototype, it is likely that dynamic strength increases are quite small; but there is little reason to expect differences between one-directional and alternating load effects in the material, since pore-pressures, thixotropy and remolding effects are not involved. With regard to the model clay core, the increase of dynamic strength over static strength measured in one-directional tests should apply equally well to alternating loadings because for this highly super-saturated, semi-fluid kaolin mixture there should be no difference in pore pressure effects, and remolding or thixotropic effects are negligible.[15] The analysis presented by Mr. Ambraseys to support his hypothesis regarding loss of shear strength of the core during earthquakes has no significance because it is based on the erroneous assumption that the water content of all samples before testing was 125 per cent. Actually, as was stated in the paper, the initial water content was made higher than 125 per cent in all cases in an effort to provide the desired shear strength at the time of the test, it being recognized that the shear strength would increase with time due to consolidation.

The situation is somewhat different with respect to the prototype clay core, for here it is believed that thixotropic effects might be present to some degree, depending upon the material involved. However, the writers are not aware of any evidence that remolding losses will be significant in compacted clays, nor is it likely that earthquake distortions could cause any significant pore pressures in this carefully compacted material. Thus, the loss in strength which might be expected in alternating loadings should only be due to thixotropy, and the gain in strength associated with dynamic loadings (as measured in one-directional tests) would probably more than offset this thixotropic loss.

The writers agree with Mr. Ambraseys that the increase of strength exhibited by the cores of dams E and F over the intended value is due to consolidation. It was not intended to imply that the increases were due to dynamic effects, as he apparently has inferred was the intent. This increase in strength represents a lack of similitude in the model which was entirely unintentional but unavoidable.

The writers also concur with Mr. Ambraseys with regard to the importance of foundation conditions in the over-all strength of the dam. However, these models were designed to evaluate the stability of a particular prototype structure which happened to be founded on solid rock, consequently the models were constructed on a solid base. It is therefore apparent that these observed results may be applied to other foundation conditions only with caution.

Finally, with regard to the length of the reservoir behind the dam, it should be noted that the tests were conducted so that the initial impact caused the dam to move away from the reservoir. The tests were deliberately designed this way because the principal objective of the study was to observe tendencies toward slip failures in the clay core and these tendencies were accentuated by

downstream accelerations. In this case the length of the reservoir manifestly would have little to do with failures in the clay core, and lack of similitude in this respect was considered unimportant (although it is granted that downstream settlement effects might have been slightly more pronounced if the reservoir had been somewhat longer).

In conclusion the writers wish to thank both Mr. Spielman and Mr. Ambraseys for the time and effort they spent in commenting on this paper.

REFERENCES

15. Seed, H. B. and Chan, C. K., "Thixotropic Characteristics of Compacted Clays," Proc. ASCE Vol. 83, No. SM 4, November 1957, Part 1.

FRICTIONAL RESISTANCE OF STEEL H-PILING IN CLAY[a]

Closure by Eben Vey

EBEN VEY,[1] M. ASCE.—The writer wishes to thank Messrs. Wu, Menard, McNeill and Lundgren for their very interesting and stimulating discussions. Their comments have added greatly to the value of the paper. The curves which Mr. Wu gives, showing the total load transferred from pile to soil, certainly are of interest and in fact were given in the final report (unpublished) of the project. However, in attempting to evaluate the actual soil shear resistance mobilized around the pile in relation to laboratory soil shear strength measurements, the smallest increments possible along the pile length were used. These, of course were the distances between gages. The writer believes that smaller elements would have been even more desirable because the strain gage data could then have been compared with more uniform soil data. Averaging the data over longer lengths of pile as Mr. Wu suggests would, the writer believes, make the comparison less accurate.

It is agreed, as Mr. Wu points out, that assuming no friction between pile flanges and soil is an extreme condition and is indeed as Messrs. McNeill and Lundgren state "no minor assumption." It may be well to restate the specific conditions for which the data indicated this extreme condition to exist.

1. Clay soil whose natural water content was close to the plastic limit.
2. Loads large enough to mobilize the ultimate shear strength of the clay.
3. Short term loading.

It is believed that for some practical pile loadings the condition might be closer to shear on the full perimeter, even in this type of soil, but the shear stress developed in the soil would then be considerably less than the full soil strength.

The laboratory tests on friction between steel and clay were only qualitative. It did not seem necessary at the time to pursue these tests further, after it was demonstrated that the shear strength of the clay was in excess of the frictional resistance between the steel and clay. The phenomenon observed was that friction existed between steel and the remolded clay up to a certain critical load. It dropped suddenly to zero when the load, which was less than the shear strength of the clay, was reached. A similar action was therefore assumed to take place between the outside of the pile flanges and the clay. In the area between the flanges there was probably some arching in the clay which, together with the added surface area, was sufficient to develop the full shear strength.

As Messrs. McNeill and Lundgren point out, the half perimeter assumption

---

a. Proc. Paper 1160, January, 1957, by Eben Vey.

1. Prof., Illinois Inst. of Technology, Chicago, Ill.

might be in considerable error if applied to other soils. It may be recalled that shear on the full perimeter was assumed in the lower regions of the pile where the soil was distinctly granular. The condition illustrated in Fig. 1 by Messrs. McNeill and Lundgren is a granular soil $\phi = 28^{0}$, c = 0.1 ton/sq. ft. It is to be expected that shear on the full perimeter (case A) would apply. In this case, however, the problem is complicated by the increased effects of remolding, reconsolidation and pore pressures, thus making it much more difficult to get accurate comparable shear measurements in the laboratory.

It is not clear in Mr. Menard's discussion, exactly how the pressiometer values were converted to soil shear strength. At depths of more than a few feet it would seem to the writer that the shear pattern around the pressurized cylindrical volume might be quite complicated and this coupled with the probability of high pore water pressures for short periods of loading would make it very difficult to develop a relationship between shear strength and pressiometer readings.

There are other data, which would indicate that the clays in this general area can be sampled by conventional methods without serious loss of strength. Slips which occurred along the banks of cuts on the Congress Street Expressway, during construction, have made it possible to compute actual soil shear strength values. These have been found to compare very favorably with shear values obtained from unconfined compression tests on Shelby tube samples, taken prior to excavation.

Some years ago the writer made a series of in-place Vane tests[2] in Chicago clay to a depth of approximately 50 ft. and compared the shear values obtained with unconfined compression test results on samples taken with a split barrel sampler, which in the writer's opinion causes no less sample disturbance than the Shelby tube method. The correlation in this case was also quite good.

The pressiometer which Mr. Menard describes appears to be a device which might be very useful in determining soil restraining pressures on underground structures such as flexible tunnel linings. But the writer cannot agree, on the basis of the information given in Mr. Menard's discussion, that the results of his tests prove that there is serious loss of strength due to sampling in Chicago clay.

---

2. Soil Shear Tests by Means of Rotating Vanes by E. Vey and L. Schlesinger Proceedings of the Twenty-Ninth Annual Meeting of the Highway Research Board, December, 1949.

FIELD EXPERIENCE WITH CHEMICAL GROUTING[a]

Closure by Milos Polivka, Leslie P. Witte, and John P. Gnaedinger

MILOS POLIVKA,[1] A. M. ASCE, LESLIE P. WITTE,[2] and JOHN P. GNAEDINGER,[3] J. M. ASCE.—The writer (Polivka) wishes to express appreciation to Messrs. J. P. Elston and G. A. Kravetz for their discussions which in themselves are valuable contributions to the field of chemical grouting. Of special interest is Mr. Elston's description of the two field applications of silicate grouts. The writer (Polivka) agrees with Mr. Elston's statement that each grouting problem has to be considered as a separate entity and that, to be successful, field research at the site should be conducted.

Mr. Kravetz questions the writer's (Polivka) statement that some silicate grouts form "permanent" gels. The word "permanent" as used means that the gel will last as long as needed, it was not the intent to imply that these gels are of geological permanency.

The writer (Polivka) agrees that the statement that "chemical grout can be injected into any soil into which water can be injected" might be misleading since the silicate grouts have a somewhat higher viscosity than water. A field trial employing water however, would clearly determine whether chemicals could be practically injected. Final field tests should be made with the chemical which is to be used.

The writer (Witte) appreciates Mr. J. P. Elston's valuable contribution to the literature of his chemical grouting experiences of foundations for warehouse column footings at David Dam, Arizona, the sandstone rock near Glen Canyon Dam, and in the grouting of the joints of the outlet conduit at Heart Butte Dam. In regards to the grouting at Heart Butte, Mr. Elston questions the statement, "No appreciable amount of cement could be injected . . ." This is considered to be a minor point. However, the objective was used to point out that portland cement grout did not penetrate the fine sands and did not effectively seal the joints, and Mr. Elston indicates concurrence in his conclusion b, paragraph 3, Part C of the discussion.

---

a. Proc. Paper 1204, April, 1957, by Milos Polivka, Leslie P. Witte and John P. Gnaedinger.

1. Asst. Prof. of Civ. Eng., Univ. of California, Berkeley, Calif.
2. Materials Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.
3. Pres., Soil Testing Services, Inc., Chicago, Ill.

COMPACTING EARTH DAMS WITH HEAVY TAMPING ROLLERS[a]

Closure by Jack W. Hilf

JACK W. HILF,[1] M. ASCE.—The discussions by J. D. Hodgson and C. Y. Li are appreciated. It is interesting to note that both writers conclude that too much energy must have been expended by using the same roller, thickness of lift, and number of passes on a variety of soils.

Mr. Hodgson suggests that other types of rollers than the sheepsfoot roller would have permitted deeper layers, thereby reducing the energy per cubic foot applied to the soil. He recognizes, but dismisses as being of little significance, "small variations (up to 10 per cent)" in density from top to bottom of a layer, which is the consequence of surface compaction of thick lifts by rollers other than the tamping type. The writer believes it undesirable to accept "up to 10 per cent" variation of density within a compacted layer of the water barrier portion of an earth dam. Density gradients within a layer are undoubtedly accompanied by much greater permeability gradients. Flow net analysis shows that horizontal layers of different permeabilities within the embankment have the effect of raising the phreatic line and reducing the stability of the dam for the steady-state condition.

Mr. Hodgson's observation that the optimum moisture content for the rollers used was probably considerably lower than that obtained in the laboratory is correct. The data show also that the peak density of the roller curve is generally several pounds per cubic foot higher than the peak density of the laboratory curve. For the design criteria used, this condition is a desirable one rather than indicating wasted effort. For dams susceptible of pore-water pressure development during construction, the objective in Bureau of Reclamation procedure is to obtain full densities comparable to the laboratory maximum density at a placement water content somewhat dry of the laboratory optimum. The only way to accomplish this is to specify a compactive effort greater than the laboratory effort.

The writer agrees with Mr. Hodgson that in granular materials (such as are commonly used in pervious portions of earth dams) efforts to improve the bond between layers by harrowing or similar means are generally not warranted. His request for definite information on bonding of layers of fill can be answered, at least partially, by examination of the published reports of an outstanding study of compaction made by the Waterways Experiment Station of the Corps of Engineers.(1) Report No. 7, June 1956, page 35, includes the following statement as one of the results of compacting a lean clay, CL, (LL = 36 per cent, PI = 15 per cent) by sheepsfoot roller and by rubber-tired roller:

---

a. Proc. Paper 1205, April, 1957, by Jack W. Hilf.

1. Superv. Civ. Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

"For the material compacted by the rubber-tired roller, a poor bond between lifts was observed at all water contents, and laminations within the lifts or a flaky structure were observed for the material compacted on the wet side of optimum. For the material compacted with the sheepsfoot roller, a good bond was obtained between lifts at all water contents; however, laminations were observed in the material compacted on the wet side of optimum but to a smaller degree than that observed for material compacted with the rubber-tired roller."

Similar results were obtained in the field and in the laboratory by the Bureau of Reclamation.(2) There appears to be considerable evidence that poor bond between layers is characteristic of compaction of cohesive soils by pneumatic or other surface-type rollers.

Mr. Hodgson's conclusions on the significance of his observation of an excavation at Adaminaby Dam in Australia does not appear to be warranted. The use of the sheepsfoot roller does not in itself constitute a guarantee that layers of compacted soil will be indistinguishable; difference in soil type, color, or water content of the loose lifts will remain after compaction. The compacting equipment cannot substitute for nonhomogeneity of material; however, it should insure that the layers are well bonded. The ripping of smooth surfaces caused by hauling equipment before fresh material is placed is considered to be a good method of facilitating bond between lifts of cohesive soil. The writer believes that energy used in achieving that result is beneficial rather than wasteful in dam construction.

Mr. Hodgson's request for information on construction pore pressures in nine of the dams reported is answered briefly in Table 4. Quantitative information on some of these dams was given by Daehn in 1951.(3) A detailed treatment of this subject is beyond the scope of this discussion, but it should be pointed out that the percentage of air voids in the soil mass is only one factor in the development of construction pore pressures. For example, the compressibility of the soil is an even more important factor.

Table 4

Dam	Construction Pore-Water Pressure
Green Mountain	Appreciable
Deerfield	No instruments installed
Angostura	No instruments installed
Heart Butte	Small
Anderson Ranch	Appreciable
Granby	Appreciable
Boysen	Appreciable
Willow Creek	No instruments installed
Vermejo	No instruments installed

As indicated by the footnote on page 1, the writer's paper was prepared in 1955, hence, it did not consider specifically the interesting concepts contained in Mr. Li's Paper No. 862, published in SM 1 January 1956. The latter's idea of "stage" compaction using first a light pressure roller followed by a heavy roller is believed to be theoretically sound. Variation of contact of sheepsfoot rollers as well as variation of weight for different soils, water contents, and

thicknesses of lift also appear to be conducive to efficient compaction.

The advantages and disadvantages of varying the size of tamping feet were discussed in the original paper under the heading "Walkout". The writer believes that the cost of requiring several rollers for stage compaction or several sets of tamper heads to meet varying soil or moisture conditions of the borrow material will be appreciably greater than the cost of rolling with one heavy roller. In that connection and to answer Mr. Li's question on the cost of compaction of a variety of soils by the heavy tamping roller, a review was made of the contract costs of the dams reported in the author's paper.

For the dams listed in Table 1, the cost of earthfill in dam embankment including the operations of spreading; moistening; harrowing, if necessary; and compacting each 6-inch (compacted thickness) layer of soil with 12 passes of the heavy tamping roller varied from 8 cents a cubic yard to 35 cents a cubic yard with a weighted average of 14.1 cents a cubic yard. The actual cost per pass of the roller is estimated to be 0.35 cents per cubic yard, based on rental rates and average production output. The total cost of compacting 75 million cubic yards by this method was $10,600,000 or 6.7 per cent of the total cost of the dams listed in Table 1.

It is interesting to note that in the bid opening for Fort Cobb Dam, Oklahoma, on January 9, 1958, the low bid for 3,300,000 cubic yards of earthfill was $0.05 a cubic yard which constitutes only 5 per cent of the total bid price. The average of the three low bidders for this item was 0.08-2/3 cents. It is believed that little, if any, saving could have been accomplished on this project had special test sections been made to determine the optimum rolling area of tamping feet, number of passes, and thickness of lift; and had the specifications been changed accordingly. There would still have been no assurance that changing moisture conditions during construction would not require changes in the specified method of compaction.

It is concluded that the actual cost of compaction of dam embankments is small in comparison with its importance and with the total cost of the work. Until other means of compacting cohesive soils are found that will match the performance of the heavy tamping roller and will permit more flexibility at no greater overall cost, there appears to be ample justification for continuing the use of heavy sheepsfoot rollers for compacting the water-barrier portion of earth dams.

REFERENCES

1. Soil Compaction Investigation Reports No. 1-8, Technical Memorandum No. 3-271, Waterways Experiment Station, Vicksburg, Mississippi, 1949-1957.

2. J. W. Hilf, Discussion of the paper, "Effect of Tire Pressures and Lift Thicknesses on Compaction of Soil with Rubber-tired Rollers," by W. J. Turnbull and C. R. Foster, Proceedings, Joint Meeting, ASTM Committee D-18 and Sociedad Mexicana de Mecanica de Suelos, December 1957 (scheduled for publication by American Society for Testing Materials, Philadelphia, Pennsylvania, in February or March, 1958).

3. W. W. Daehn, "Pore Pressure in Design" Part I of "Implications of Pore Pressure in Design and Construction of Rolled Earth Dams." Transactions Fourth Congress on Large Dams, New Delhi, 1951, Volume 1, pages 259-270.

A REVIEW OF THE THEORIES FOR SAND DRAINS[a]

Discussion by S. J. Johnson
Closure by F. E. Richart, Jr.

S. J. JOHNSON,[1] M. ASCE.—A singularly penetrating analysis of the influence of the various factors affecting the theoretical aspects of the design of sand drains has been presented by the author. His thorough review of Terzaghi's theory of consolidation and of its extension, mainly by Barron, to the theory of consolidation by radial drainage or by a combination of radial and vertical drainage, will be appreciated by those engineers concerned with the use of vertical sand drains for stabilization of soils through consolidation. His review of numerical methods of solution of consolidation phenomena and his illustration of means of taking into account variable loading and soil characteristics will be appreciated and applied to problems of consolidation due to vertical flow as well as to problems of consolidation including radial flow and is of general interest beyond the particular field of sand drains. The discussion on effect of diameter of sand drains pointedly illustrates how easy it would be to over-assess the importance of large diameters. However, the very small diameters referred to are presumably used for illustration only by the author. The writer interprets this discussion as not meaning to imply that such small diameters would be practicable for sand drains installed by methods currently used.

It is necessary for some purposes to know the theoretical excess pore water pressures at specific locations when piezometers are used for field control purposes and for evaluation of the coefficient of consolidation. These can be simply prepared from the property that, for the equal strain theory, the plot of $\log \frac{u}{u_o}$ versus time factor is a straight line, as illustrated on Fig. 1 attached hereto, also that the same is true for average excess pore water pressures as plotted on Fig. 2. This is not true for average consolidation due to vertical flow, which is plotted in a similar manner on Plate 3. Figs. 1 and 2 are obtained as follows:

Eqs. (15) and (16) can be written, for any specific sand drain installation and any point r from the drain, as:

$$\frac{u_r}{u_o} = \frac{4}{d_e^2 F(n)} \left[r_e^2 \log_e \frac{r}{r_w} - \frac{r^2 - r_w^2}{2} \right]$$

$$= \text{constant} \times \mathcal{E}' = \text{constant} \times \left[-\left(\frac{8 T_h}{F(n)} \right) \right]$$

a. Proc. Paper 1301, July, 1957, by F. E. Richart, Jr.

1. Engr., Moran, Proctor, Mueser & Rutledge, New York, N. Y.

Therefore,

$$\log_e \frac{u_r}{u_o} = C_1 + C_2 T_h$$

where

$$C_1 = \log_e \left[\frac{4}{d_e^2 F(n)} \left\{ r_e^2 \log_e \frac{r}{r_w} - \frac{r^2 - r_w^2}{2} \right\} \right]$$

and

$$C_2 = - \frac{8}{F(n)}$$

This is the equation of a straight line when $\frac{u_r}{u_o}$ is plotted to a log scale and T_h is plotted to an arithmetic scale. Fig. 1 is this relationship at the external boundary where $r = \frac{d_e}{2}$.

The relationship between the average excess pore water pressure from $r = r_w$ to $r = r_e$ and the time factor T_h is given by the author's Eq. (16) as:

$$\frac{\bar{u}}{u_o} = \epsilon^{n} \cdot \epsilon^{-\frac{8T_h}{F(n)}}$$

from which

$$\log \frac{\bar{u}}{u_o} = - \frac{8}{F(n)} T_h$$

This is also the equation of a family of straight lines, as plotted on Fig. 2. Plots of this type shown on Figs. 1 and 2 are readily constructed and are useful in analysis of field observations as well as in initial design.

In conclusion, the writer wishes to compliment the author for his interesting and intensive paper.

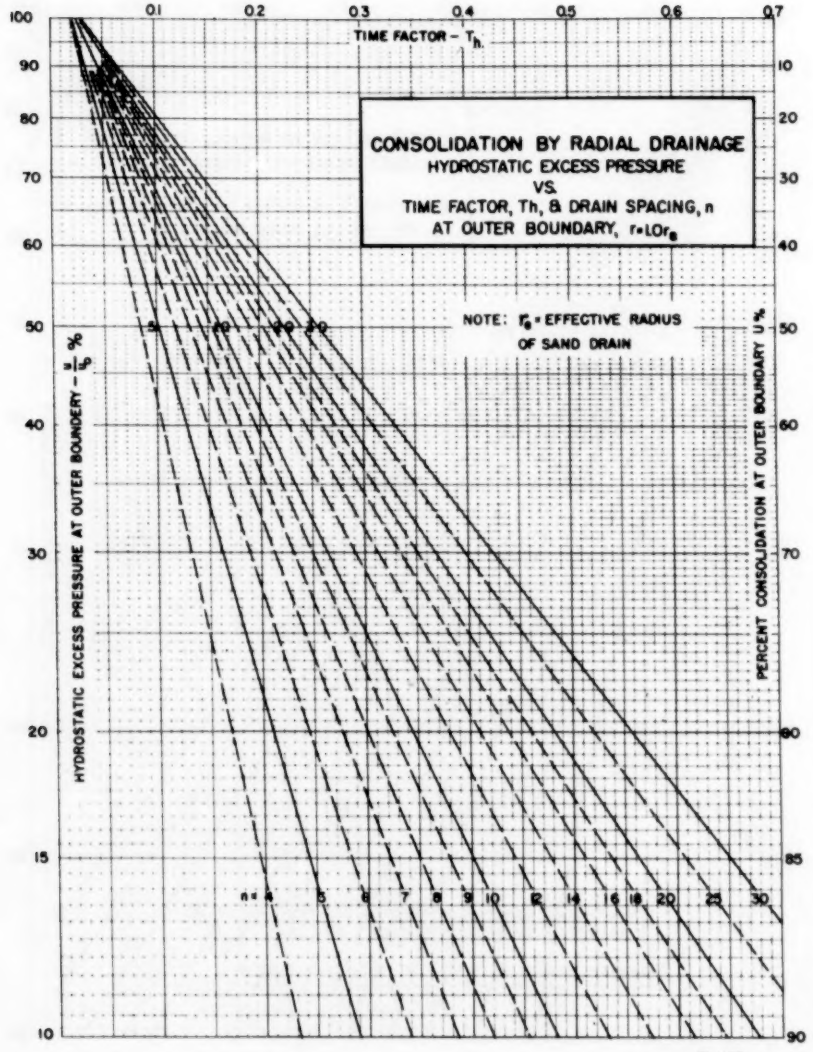
F. E. RICHART, JR.,[2] M. ASCE.—Thanks are extended to Mr. Nishida and Mr. Johnson for their interest in the paper and for their generous comments.

Mr. Nishida notes that a study made in Japan found the time for consolidation is proportional to the 2.5 power of the well spacing. By use of Eq. (16) the time required for a particular degree of consolidation is proportional to the 2.76 power and 2.38 power of the well spacings when comparing times for 5 and 10 ft. spacings, and 20 and 40 ft. spacings, respectively. Thus, the use of the 2.5 power of the well spacing agrees quite well with results obtained from Barron's "Equal Strain" condition of consolidation.

With regard to Mr. Nishida's request for information on the most suitable well spacing from the practical point of view, this would necessarily depend upon local soil and construction conditions. The terms suitable and practical imply that the spacings must be feasible and economical in both time and money. From a recent study of existing sand drain installations(20,21) the well spacings most frequently used in 83 installations were 8 to 10 feet.

Mr. Johnson is correct in his interpretation of the discussion of the effectiveness of small diameter "ideal" wells. The writer was not proposing the use of actual sand drains of 1 or 2 inches in diameter. The discussion

2. Prof. of Civ. Eng., Univ. of Florida, Gainesville, Fla.


CONSOLIDATION BY RADIAL DRAINAGE
HYDROSTATIC EXCESS PRESSURE
VS.
TIME FACTOR, Th, & DRAIN SPACING, n
AT OUTER BOUNDARY, r=10r_e
NOTE: r_e = EFFECTIVE RADIUS OF SAND DRAIN
TIME FACTOR - T_h
HYDROSTATIC EXCESS PRESSURE AT OUTER BOUNDERY - u/u_o %
PERCENT CONSOLIDATION AT OUTER BOUNDARY U %
n = 4 5 6 7 8 9 10 12 14 16 18 20 25 30
PLATE

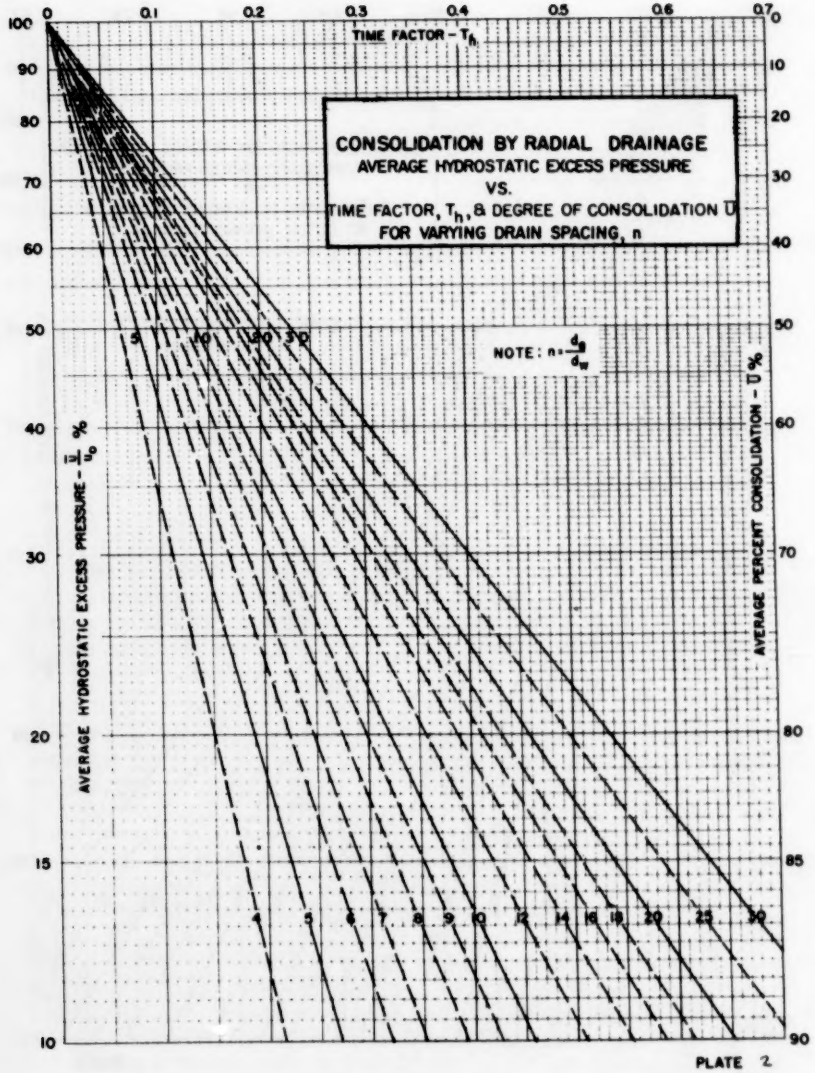

TIME FACTOR - T_h
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
CONSOLIDATION BY RADIAL DRAINAGE
AVERAGE HYDROSTATIC EXCESS PRESSURE
VS.
TIME FACTOR, T_h, & DEGREE OF CONSOLIDATION $\bar{U}$
FOR VARYING DRAIN SPACING, n
NOTE: $n = \frac{d_e}{d_w}$
5
10
20
30
4
5
6
7
8
9
10
12
14
16
18
20
25
30
AVERAGE HYDROSTATIC EXCESS PRESSURE - $i = \frac{I}{I_0}$ %
AVERAGE PERCENT CONSOLIDATION - $\bar{U}$ %
100
90
80
70
60
50
40
30
20
15
10
0
10
20
30
40
50
60
70
80
85
90

PLATE 2

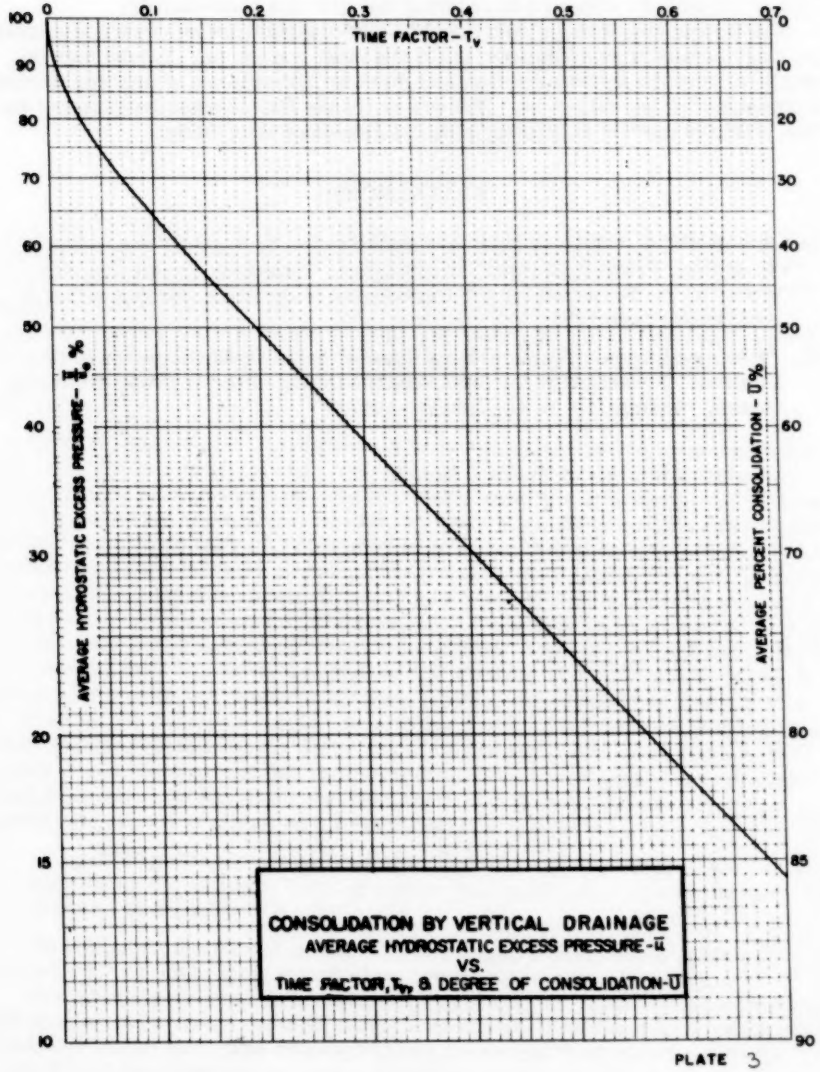

TIME FACTOR - T_v
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
AVERAGE HYDROSTATIC EXCESS PRESSURE - $\frac{u}{u_0}$ %
AVERAGE PERCENT CONSOLIDATION - $\bar{U}$ %
100
90
80
70
60
50
40
30
20
15
10
0
10
20
30
40
50
60
70
80
85
90
CONSOLIDATION BY VERTICAL DRAINAGE
AVERAGE HYDROSTATIC EXCESS PRESSURE - $\bar{u}$
VS.
TIME FACTOR, T_v, & DEGREE OF CONSOLIDATION - $\bar{U}$

PLATE 3

was intended to point out that even if the smeared zone around a 10 or 12 inch diameter actual drain well restricted drainage until the time for consolidation was comparable to that for a 1 or 2 inch diameter "ideal" well, that the drain well would still be effective in reducing the time for consolidation.

The figures prepared by Mr. Johnson showing pore pressure on a log scale vs. time factor on an arithmetic scale are most convenient. Values may be obtained from the figures readily, and there is little chance of drafting errors in constructing the diagrams. The writer thanks Mr. Johnson for pointing out this effective method of plotting consolidation-time information.

REFERENCES

20. Report No. 1, "Study of Deep Soil Stabilization by Vertical Sand Drains" by Moran, Proctor, Mueser, and Rutledge, Consulting Engineers, New York, N. Y. Prepared for the Department of the Navy, Bureau of Yards and Docks, June 1956.

21. "Review of Uses of Vertical Sand Drains" by P. C. Rutledge and S. J. Johnson. Presented at the 36th Annual Meeting of the Highway Research Board. Washington, D. C., January 1957.

THIXOTROPIC CHARACTERISTICS OF COMPACTED CLAYS[a]

Discussion by D. P. Krynine

D. P. KRYNINE,[1] M. ASCE.—This paper is a laboratory investigation in the rheology (i.e. scientific study of deformations) of compacted clays, excellently executed and clearly presented. Though the measurements were made with instrumentation of high precision, the conclusions are qualitative in character only. Several of the results obtained could be anticipated from the work of others. No field observations by the authors are reported.

Definitions

In 1935 Freundlich(3) described the isothermal reversible transformation of a true sol to a true gel and vice versa as a "limiting case of thixotropy." According to him "thixotropy also includes some related phenomena." Pastes of clay including bentonite, for example, "may show an extremely pronounced thixotropic behavior." In a dissertation by one of his students(11) published only three years later, it is stated that the concept of thixotropy was "gradually extended to all those systems which are liable to a temporary change of consistency upon mechanical deformation." This is practically the same point of view that prevails at the present time (1958). Thus the concept of thixotropy included in the "Report on the Rheological Nomenclature"(4) by two eminent rheologists in reality does not advance anything different from Freundlich's concept in its widened version. It is to the credit of the authors of this paper that their paraphrase of the definition of thixotropy given in the Report is clearer and more acceptable to the engineer than the original version. The definition given by Skempton and Northey(1) and quoted below is brief and clear, but leaves unexplained the term "bonding forces":

> . . . "Thixotropy results from the gradual rearrangement of the particles under the action of bonding forces, into position of increasing mechanical stability."

The thixotropic process consists of two steps, a destructive and a constructive; and the preceding definition covers the constructive step.

The rheologists distinguish thixotropy of shape (or volume), and thixotropy of rheological properties. Soil strength should be considered as a rheological property of the soil. The rheologists use the terms "recovery" and "restoration" for the constructive step of these two kinds of thixotropy. Skempton and Northey and the authors use the term "regain." The writer believes that these three terms may be used interchangeably in engineering usage.

---

a. Proc. Paper 1427, November, 1957, by H. B. Seed and C. K. Chan.

1. Cons. Engr., Berkeley, Calif.

Since Freundlich's work not very much was written on thixotropy during the interval 1935-1948. Besides the references given in the paper, references (12) and (13) also should be included in the bibliography.

"Stirred" State of Compacted Clays

A substance that at proper occasions shows thixotropic effects, is called "thixotropic substance" by the rheologists. As stated, clays have been found to be thixotropic substances by Freundlich;(4) also Skempton and Northey (Fig. 2 of the paper) found thixotropy with basic clay minerals.* A thixotropic substance, i.e. clay, if subjected to a deformation process "exhibits a change in rheological properties,(4, pp. 41 and 42) particularly partial loss of strength and occasionally change in shape and/or even volume. Rheologically speaking, it is then in the so-called "stirred" state(4, p. 43) which is not the state of equilibrium, however, and the substance in this state is "always subject to a gradual change "(4, p. 43), particularly thixotropy. Compacted clays are in this "stirred" state just by virtue of compaction, hence rheological effects, particularly thixotropy, are to be expected in them. It appears, then, that special experiments set up with the special purpose to prove the thixotropic properties of the compacted clays are not essential but, as an additional proof may be useful.

Clay Structure and Structure Building Factors

The clay strength is controlled by its structure, predominantly its microstructure, i.e. the mutual arrangement of very small particles. Macrofeatures such as inclusions or fissures may influence the strength of the clay mass only locally. The clay particles are tiny crystals, mostly platy, built of exceedingly thin atomic sheets. When dry, a clay mass is practically electrically neutral and does not show attractive properties. When placed in water, however, individual clay platelets become negatively charged and individual atoms constituting the clay material are either positively or negatively charged; these are cations and anions, respectively.

The negative charge of the clay particles in water is the reason for what is called clay activity. The manifestations of the clay activity are at least threefold: (a) the clay particle may attract cations of aluminum, iron, sodium, potassium and other elements that may constitute clay and are swarming in the clay suspension around the clay particles; (b) the clay particle may attract water from the ambient suspension; and (c) the clay particle may exchange the attracted cations against those swarming around especially if some electrolyte solution is added to the suspension (e.g. common salt solution). This is the so-called base exchange or, more appropriately, "cation exchange."

So far as attraction of water by the clay particles is concerned it should be remembered that the water molecules are dipoles, i.e. have two poles and may be visualized as exceedingly short flexible strings carrying a positive charge at one end and a negative charge at the other. Thus a water molecule may be attached by its positive end to a clay particle and by its negative end

* Besides kaolinites, illites, and montmorillonites chlorites are to be considered basic clay minerals. Particularly, black chlorite adobe may be found in the San Francisco Bay area (information by Woodward, Clyde, Sherard and Associates, Oakland, Calif.)

to a cation or another water molecule. It is understandable that in a row of molecules attached to each other, starting at the surface of a particle the attraction force gradually decreases. It is at a maximum just close to the clay particle where it is heavily reinforced by the attractive van der Waals-London forces (that are operative in all kinds of matter). At a small distance from the clay particle these forces practically vanish.

Thus in a saturated clay a portion of its water is strongly attached to the clay particles, thus representing an exceedingly thin but highly rigid framework of the clay mass. Inside of this framework there are clay particles, and outside of it, there are, first, a not so strongly attracted but nevertheless immobilized layer of water, and, around this layer, free water. A clay mass with a high degree of saturation (as most of the experimental clay discussed in this paper) is in reality a clay suspension and may be treated as such. It is of variable rigidity, helps to keep the particles together and fills the voids between them.

Clay particles, anions, cations, water in different states of attraction and, last but not least, the electrical forces constitute the structure of the compacted clay. The fruitful idea of considering the electrical forces, as a component of any clay structure belongs to Lambe.[15] Using it, explanation of thixotropic processes becomes relatively simple.

Residual and sedimentary clays are built in two different ways. A residual clay deposit is formed essentially by physical and chemical destruction of the parent rock, but is subject to thixotropic changes if compacted. The thixotropic process will be intensified if the compacted clay is loaded or otherwise "stirred" (to use the rheological term).

A sedimentary clay is built by the process of sedimentation when clay particles settle down and are accommodated in their places by two mighty forces, namely gravity and mutual attraction when in close packing. Flocculation is a particular case of the sedimentary clay building process when the repulsing forces acting on the particles charged with electricity of the same sign are overcome by the van der Waals-London forces. Flocculation is generally accompanied by the base exchange. Besides gravity and mutual particle attraction there are many other structure-building factors in a sedimentary clay, such as the nature of the attracted cations, water-sorption characteristics, particle orientation and spacing, capillary tensions (especially in the partly saturated soils) etc.[14]

When compacted clay is mechanically disturbed (e.g. by loading), some structure-building factors may decrease in intensity or completely vanish from the scene; some other factors may even increase in intensity. The two clay structure-building factors that always survive a mechanical disturbance are gravity and attraction forces (or more broadly, clay activity); thus these two factors are responsible, fully or partly, for recovery and restoration processes. Attraction forces in clay may be operative only if there is enough water around the particles. All writers on thixotropy emphasize this requirement. Freundlich, for example, states[3, page 16] that "thixotropy implies that the systems of particles enclose a fairly large amount of liquid, i.e. they must be loosely packed." In reality, a loose structure is not a cause of thixotropy, but a contributing factor. Thus, for example, one of the reasons why the montmorillonites show more thixotropy than the kaolinites is the loose packing of their atomic sheets that permits water to enter a particle, and this, of course, in connection with their strongly developed primary and secondary surface-area.

Thixotropic and Partially Thixotropic Clays

Thixotropy is not an intrinsic property of a clay; and clay may be more or less thixotropic according to the degree of clay activity in the presence of water, and efficiency of other structure building factors in operation. Generally a substance may be called thixotropic if it exhibits thixotropic behavior whether under any or only under some definite circumstances, as, in some other wording, is stated in item (2) of this discussion. Particularly, so far as compacted clays are concerned, they are all potentially thixotropic. From this point of view a subdivision of clay materials does not appear justified, and Fig. 1 if applied to clays is a particular case of Fig. 8. The level marked "hardened" in this figure may coincide with the level marked "undisturbed" or be even higher. The latter case corresponds to the building up of a completely new clay structure by the structure-building forces. A well known example is pile driving into clay of low sensitivity with considerable increase in shearing strength of the material around the pile; strictly speaking, this is no longer thixotropy.

Thixotropic Strength Ratio

As may be seen from Fig. 12, top, this ratio may not be sensitive enough; and, besides, it may be misleading as in the following example. Consider Fig. 12 in two instances.

(a) Samples at 19 and 16 per cent of molding water content, one on the wet side and the other on the dry side of optimum. Whereas the respective thixotropic strength ratios are 1.3 and slightly over 1, the increase in the value of the stress causing a 10 per cent strain is the same in both cases, namely, 0.6 kg. per sq. cm. as measured on the diagram;

(b) Samples at 18 and 19 per cent of molding water content, both samples being on the wet side of optimum. The increases in strength are 1.1 and 0.6 kg. per sq. cm. respectively, whereas the respective values of the thixotropic strength ratios are 1.25 and 1.3. From an engineering point of view, the effect of thixotropy on the 18 per cent water-content sample is more pronounced than on the 19 per cent water-content sample, whereas the thixotropic strength ratios inform the engineer otherwise.

In actual practice 90 per cent compaction is used. The compaction test (Proctor) curve in Fig. 12, bottom, is somewhat too short and does not give complete information on such compaction in this particular case.

Increase in Thixotropic Effects in Normal Strength Tests

Fig. 15 is an interesting chart. Only one glance at it is necessary in order to appreciate the thoroughness of the laboratory work involved; further examination confirms this first impression but suggests that a number of readings were perhaps superfluous.

Correctness of Fig. 12 may be proved by a rheological analogy, also based on laboratory tests.(4, V-42, Fig. 9) The modified Fig. 15 and Fig. 9 from ref. 4 are reproduced in Figs. A and B of this discussion.*

* It should be noted that the rheologists often interchange the terms "flow" and "deformation." It should also be noted that clay shown in Fig. 15 had a degree of saturation of 95%; it was practically a suspension that "flowed" when deformed.

If a "stirred" thixotropic substance is acted upon by a constant shearing stress and the resulting deformations are plotted against time t, the γ - t curve "will be concave upwards (accelerated flow), and will probably finally approximate to (sic) a straight line." This is curve (a) in Fig. 9. Furthermore, "if the experiment is interrupted . . . and if after a very short interval the same stress is applied again, flow will appear again with, from the start, practically the same high value of velocity as was obtained at the end of the first experiment." This is depicted by curve (b) in Fig. 9. "Curves (c) and (d) refer to experiments repeated after larger intervals of time."

If the experiments shown in Fig. 15 were performed successively on one sample in each series shown in the figure, the horizontal axis could be considered as the time axis and the vertical one as the deformation axis. Hypothetical deformations γ and actual strains plotted on the vertical axis, Fig. 15, will be assumed to be proportional. Then the experiments should give six curves qualitatively similar to those shown in Fig. 15 but located in the reverse order, i.e. the curves at the extreme left and the extreme right of the series now corresponding to 0 and 28 days of storage, will correspond to 28 and 0 days, respectively. A time scale cannot be properly selected in this case because of the uncertainty of numerical characteristics of the experimental material. The curves in Fig. 15 are fairly straight and parallel where they should be so. The slope of these lines may differ from that of Fig. 15 according to the time scale chosen.

Retardation

It is convenient to introduce this concept at this stage of the discussion. When the load causing deformation in clay is gradually relieved, a portion of the deformation is recovered. The rate of such recovery is not constant, but gradually increases toward the end of the relieving process, i.e. the recovery is retarded. A well known case of retardation is depicted by the rebound curve in the unloading stage of the familiar consolidation test. The curve is generally concave, often with increasing curvature toward the end. Fig. (C) is an example of such a curve obtained in an actual test on silty clay. The pressure on the sample was relieved by a certain amount (call it 100 per cent). When 75 per cent of that amount were relieved, the deformation decreased 33 per cent only as shown in Fig. C.

It is probable that in such cases the cause of recovery is partly elasticity and partly thixotropy. In fact, upon application of the load on the sample its microstructure changes, including electric forces. Particularly water is torn off the particles, becomes free and is squeezed out being acted upon by the gradient developed. The clay is still active, however, and as soon as water is re-admitted back to the sample, the recovery process starts. Strictly speaking, this is a case of volume recovery, but retardation took place also in the Skempton and Northey's observation of thixotropic strength regain in five clays (Figs. 5 of both this paper and ref. 1; also page 37 of ref. 1). The rheologists are familiar with the recovery retardation in thixotropy(4, V-34). Soil mechanics investigators should pay more attention to the phenomenon of retardation that takes place in their laboratories practically every day.

It is to be noticed that Fig. 16 and the upper curve in Fig. 17 refer to the same experiment. As one may conclude from Figs. 16 and 17, an idealized

thixotropic curve on logarithmic scale is concave, degenerating into a practically straight line (similar in shape to the curves in Fig. B). This is a theoretical case of an infinitely long thixotropic recovery. The curves for Vicksburg silty clay and Pittsburg sandy clay fit this pattern. The curve in Fig. 16 would also perfectly fit this pattern if a reading at 100 min. were taken in the laboratory. The curve for Friant-Kern clay has not been considered by the writer because it is constructed from erratic and perhaps erroneous data. Not only the curve in Fig. 16 shows that the thixotropic recovery is still going on as the authors state, but this is also the case of all curves in Fig. 17. In fact, in order to show that the process of thixotropic recovery is finished, the thixotropic curve on logarithmic plot should have a horizontal tangent at the time moment when the recovery is over. In this case the final straight line on the drawing (Fig. 17) would be bent to the right.

If desired, the above considerations may be proved mathematically. Assume that the logarithmic plot and the arithmetic plot of the experimental results have identical vertical and horizontal scales (i.e. that in the latter case one unit of log t and one unit of t are represented by equal lengths). If a portion of the logarithmic plot approaches a straight line (slope m) the slope of the tangent to the arithmetic plot at time t, within the portion under consideration, will be

$$\frac{m}{t} \cdot \log_{10} e = 0.434 \frac{m}{t}$$

where e is the base of natural logarithms. The tangent to the curve plotted on an arithmetic scale would be then horizontal (which means the end of the thixotropic recovery) in two cases: either (a) if time t is very long; or (b) if m equals zero which means a horizontal tangent to the curve plotted on the logarithmic scale.

If the experimental curve is plotted on arithmetic scale, its shape will be practically as shown in Figs. 1 and 8. The tangency point will correspond to the time moment when the horizontal tangent to the curve on logarithmic plot starts.

Effect of Storage on Deformations

Fig. 18 and 19(a) and (b) represent the same set of experiments. If the curves of Fig. 18 are plotted on arithmetic scale with times as abscissas, six curves shaped like the curve in Fig. 19(a) will be obtained. In fact, the latter curve is a detail of one of those six curves. On a logarithmic plot with logarithms of time as abscissas, a series of approximately straight lines may be obtained similar to that shown in Fig. 19(b). According to the formula in item (8), the six curves shaped like the curves in Fig. 19(a) tend to have an horizontal tangent as the time between compaction and testing grows. In other words, under the given laboratory conditions after a considerable time following compaction, to each number of standard stress applications a definite maximum strain corresponds. This may be approximately true for sandy and silty clays, but probably not of the more plastic materials.

As to the difficulty of predicting the probable life of pavements from the laboratory tests of this kind the writer agrees with the authors' opinion.

Effect of Rate of Loading

The authors use the term "creep" without explaining it. In the geotechnical literature this term is sometimes used to describe the slow translational deformations of an unloaded ground surface. In this paper the authors use the term in the sense attributed to it in ref. 9 and possibly elsewhere.

With respect to the possible changes in strength of compacted clays over a period of time the writer believes that the situation is well covered by Casagrande and Wilson who state:(9, page 263)

> "The strengths of compacted soils . . . which are not fully saturated increase with time, even when the water content is kept constant. From the standpoint of earth dam design these results have important practical implications."

This opinion is shared by many field engineers.

From several cases of strength decrease with time given in this paper and ref. 9 the writer's attention was invited to the decrease in strength of compacted clays for a certain rather small time after loading. Qualitatively similar results for a short time of loading were obtained by Casagrande and Wilson (Fig. 8 of ref. 9) and by the authors (Figs. 20, 21, 22). This means that under a long term loading the clay strength first decreases and after a certain time increases to the preloading value after which it continues increasing thixotropically. The authors explain this fact by the creep deformations. The writer believes, however, that at the time of loading, the compacted clay (which is already in a disturbed condition and ready to have its strength increased thixotropically) is additionally disturbed and needs a certain time to have its preloading strength restored. This additional disturbance caused by loading may consist in the damage of the bonds and the small displacements of particles; and it is evident, in the writer's opinion, that this restoration to the preloading condition is done by the clay structure-building forces, i.e. by thixotropy.

It is pertinent to state at this stage of the discussion that the possibility of the regain of rheological properties of a material during the intervals between the "stirring" experiments is known to the rheologists.(4, V-37)

The Authors' Conclusion

In their conclusion the authors advance Fig. 26 which shows that thixotropic processes practically do not change the angle of shearing resistance of clay but influence the intercepts on the vertical axis of the diagram expressing the Coulomb equation (Fig. 26c). This result is quite natural since thixotropy does not alter the material, but essentially increases and reinforces the bonds. The writer believes, however, that the problem of the "true cohesion" and "true angle of internal friction in saturated clays" will not be definitely solved unless more efficient research tools are used, particularly physical chemistry and allied sciences.

The Writer's Conclusion

Atterberg was probably the first man who observed thixotropy in the field intelligently. This was during the construction of a railroad in northern

Sweden, when a stratum of clay with a high water content showed a semi-fluid behavior. His observation was published in 1908 in an agricultural journal;(16) and in the same year Atterberg's paper on his "limits" appeared. This work was done only eight years after Atterberg had started pioneering in soil investigations as a middle-aged chemist and agriculturist. His work is a good example of a desirable combination of laboratory soil work and field observations.

At the present time, half-a-century after Atterberg, there are still papers on soil mechanics in which laboratory and field data are harmoniously combined. An example of such a desirable paper is a study of friction piles by a (then) graduate student Mr. Reese under the direction of Professor Seed.(17) It is, however, becoming a rather widespread tendency to try to solve problems of soil mechanics without going to the field at all. It is convenient to recall here a statement by Dr. Terzaghi as follows:

> "Soil mechanics as taught and practised at present is likely to create a dangerous illusion that the processes in the natural soil strata are always, or at least usually, as clear and simple as those in the cylindrical specimens subject to investigation in the laboratory."(18)

Though a number of years have passed since these lines were written and many improvements have been made in sampling and testing, the statement is still fundamentally true; and the writer shares its sentiment wholeheartedly.

That present-day soil mechanics suffers from the lack of field information was brought up in one of the papers presented to the Fourth Conference on Soil Mechanics and Foundation Engineering, London 1957; and this is the opinion of many engineers.

The writer's opinion is that the topics discussed in a paper on soil mechanics should be connected with actual field experiments or observations. In a purely laboratory paper, such as the present one, it should be clearly shown what meaning the paper has for actual soil engineering, and how the phenomena observed in the "cylindrical samples" are reflected in actual earth masses. Without such a detailed explanation the paper may remain in the eyes of an average file-and-rank engineering reader as something abstract having no or almost no connection with the reality.

These comments represent the personal opinion of the writer and should be interpreted as such. The writer had no intention whatsoever to detract from the value of this paper, in which so much work was involved and which has been done so carefully as to be a telling example and model for other laboratory workers.

ADDITIONAL REFERENCES

11. H. L. Roeder. "Rheology of Suspensions: a Study of Dilatancy and Thixotropy," H. J. Paris, Amsterdam 1939.

12. Henry Green and R. N. Weltmann. Thixotropy. Colloid Chemistry, vol. 6 (1946), pp. 328-347.

13. Ernst Ackermann. Thixotropie und Fliesseigenschaften feinkoerniger Boeden, Geologische Rundschau, vol. 36 (1948), pp. 10-29 (based mostly on the Norwegian information).

14. T. William Lambe. Structure of Inorganic Soil, ASCE Separate 315, October 1953; also a discussion by James K. Mitchell, Trans. ASCE, vol. 120 (1955), p. 1465.

15. T. William Lambe. The Nature and Engineering Significance of the Structure of Compacted Clay. Paper presented to the ASCE Convention, February 1958.

16. A. M. Atterberg. Landwirtsch . Versuchstat., vol. 69 (1908), p. 134.

17. H. Bolton Seed and Lyman C. Reese. The Action of Soft Clay along Friction Piles, Trans. ASCE, vol. 122 (1957), p. 731.

18. Charles Terzaghi. Discussion of the Paper "Geology and Highway Engineering," Trans. ASCE, vol. 110 (1945), p. 138.

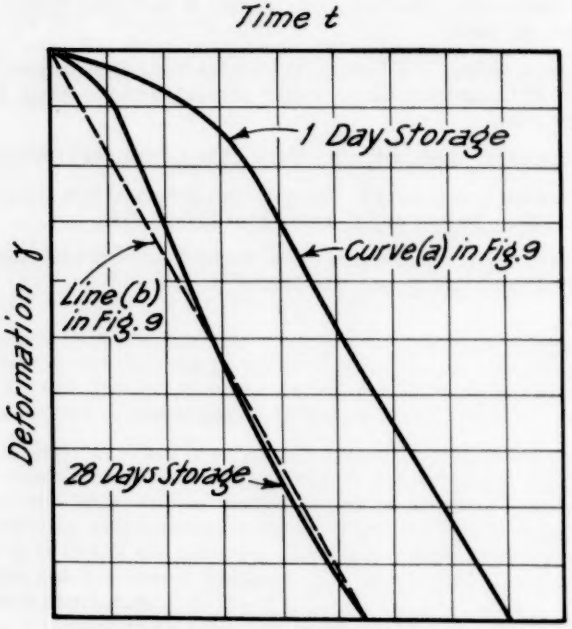


Fig.(A) Modified Fig. 15

Fig.(B). Copy of Fig.9, V-42, ref.4.

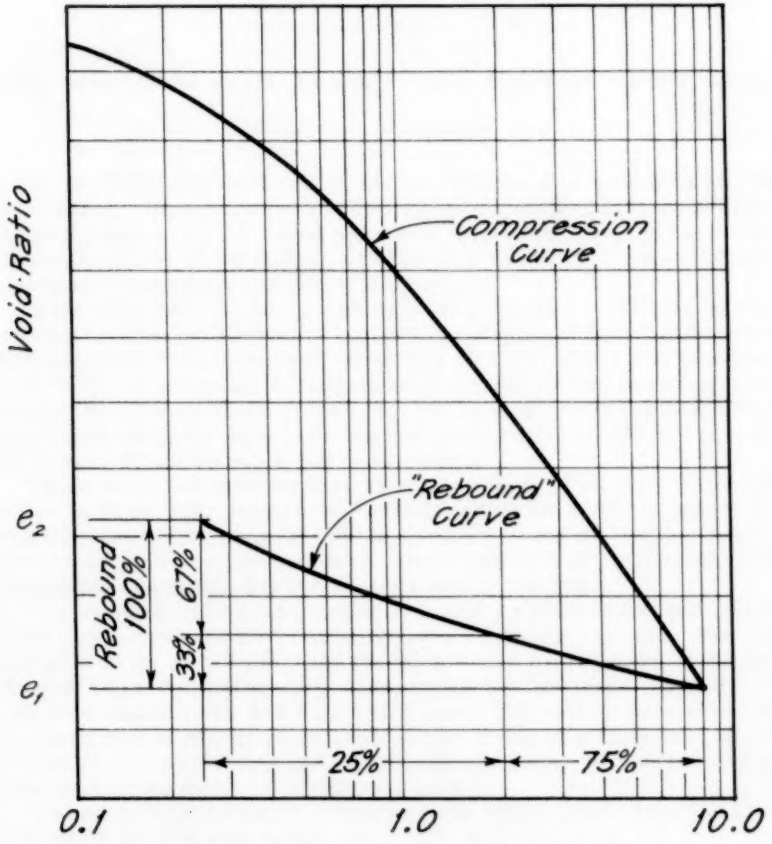


Fig. (C) Volume Recovery after a Consolidation Test. Rebound is proportional to $(e_2 - e_1)$.

LATERITE SOILS AND THEIR ENGINEERING CHARACTERISTICS[a]

Discussions by Jacques de Medina, F. L. D. Wooltorton and S. F. Gizienski

JACQUES DE MEDINA,[1] A. M. ASCE.—The paper presented by Mr. Bawa is, obviously, of great interest to soil engineers working in tropical countries. Besides presenting a very good picture of the present status of knowledge of the origin, nature and distribution of laterites and lateritic soils, the author also gives an extensive and useful bibliography.

In the discussion we desire only to bring some data concerning actual geotechnical studies of latarites of the Northern Region of Brazil and some considerations on lateritic soils in connection with the paper under discussion.

In 1953, two subgrade investigations on sections of two Brazilian federal roads—BR-22 in the state of Pará, and BR-21 in the state of Maranhão—were performed under our supervision as consulting engineer. The sections investigated were 16 and 26 miles long, respectively.

Those roads had been surfaced with only a layer of laterite, commonly known as "picarra", obtained from pits alongside the road. No quarries are available near by highway BR-22. In Northern Brazil it is customary to use laterite as a surface layer material just as "saibro"—a residual soil from the weathering of granites and gneisses—is used in the South.

The subgrade soils of that region are mostly yellowish fine silty sands with little clay, or red and brown laterite gravels with silt and sand. The former are soils of the A-2-4 group and the latter of the A-1-b group, of the Highway Research Board classification. Occasionally the "picarras" had more fines and were classified as A-2-6 or A-2-7 soils. The soil profiles often indicated layers of yellow silty clays or clayey sands at depths greater than 10 feet; therefore, some deep cuts had clayey subgrade soils. Also some profiles revealed alternate layers of laterite and sand.

The grain-size distribution of the laterites (26 samples) of the BR-22 road fell within the following range:

Sieve	% Passing
1"	90 - 100
3/4"	80 - 100
3/8"	63 - 91
No. 4	53 - 79
10	48 - 70
20	44 - 66
40	42 - 62
100	23 - 42
200	17 - 32

a. Proc. Paper 1428, November, 1957, by K. S. Bawa.

1. Engr. in Charge, Laboratory, Dept. of Highways, Federal District, Rio de Janeiro, Brazil.

A greater dispersion of grain-size distribution of laterites was found on the BR-21 road in Maranhão.

The average specific gravity of laterites was 2.77. Some of the more consistent laterites had the appearance of limonitic sandstone composed of small quartz grains cemented with iron hydroxydes. Others, less consistent, when pulverized and washed through no. 200 sieve, showed a sand fraction made almost only of quartz grains with a few limonitic concretions, and fines of brown, silty, low-plasticity material. Limonitic sandstone is used in Belém as aggregate for portland cement concrete.

From standard Proctor compaction tests we obtained dry densities varying from 130 to 124 lb/cu. ft, and optimum moisture contents from 11% to 14%.

C.B.R. tests performed on average laterite samples, compacted at 55, 25, and 10 blows per layer, according to the method developed by the U. S. Army, Corps of Engineers, gave the following CBR values—55%, 22%, and 14%, corresponding to dry densities of 133, 130, and 124 lb/cu. ft, respectively.

There was a certain amount of variation in the Atterberg limits of the laterites, depending on the preparation of samples for testing and on the resistance of concretions. However, the liquid limit of the minus no. 40 sieve fraction was generally less than 25% and the plasticity index commonly greater than 6%.

The grain-size distribution curves of the laterite samples did not quite meet the requirements of A.A.S.H.O. specifications M-147-49, type II-F, for lack of a coarse sand fraction and, sometimes, because of an excessive amount of fines. Our aim was either to select the laterites to use as stabilized base material or to correct the average laterite by adding coarse sand available from a pit near by the section of the BR-22 studied. The proportion of coarse sand to be admixed to the "picarra" was of 20%, by weight.

According to the pavement design the base course should be 15 cm (6")-thick where the subgrade was a silty sand, and only 10 cm (4")-thick where it was laterite. For the surface or wearing course a layer of a mixture of sand -cutback asphalt, 5 cm (2")-thick was suggested. The sand used was a mixture of the coarse sand already mentioned and the silty sand occurring extensively in that region. The ratio was 2 to 1, respectively. During construction the thickness of this layer was reduced to one inch.

By comparing the grain-size curves of the laterites we have studied with those reported by French engineers in Africa,[2] at Bamako-Ségou (Soudan), Dakar, Yaoundé (Cameroun) and Sénégal, we observe that the former have a more pronounced step for coarse sand, and larger fine sand fraction than the latter.

The laterite formations occur not only in the states of Pará and Maranhão but also in other areas of the Northern Region, such as the territories of Amapá, Guaporé, Acre, and Rio Branco. They have also been observed in the Central-Western Region, which includes Mato Grosso and Goiás.

The clay mineral that predominates in lateritic soil is kaolinite, a low activity mineral. It is generally observed that soils like the "saibros" of the Southeast and the "picarras" of the North practically do not expand when

2. "La route dans les Territoires Francais d'Outre-Mer", in Construction, Sept. 1955; Special issue; Dunod, Paris.

saturated as CBR samples. Considering the concept of "activity" by Skempton[3]—ratio of plasticity index to minus 2 micra soil fraction—we realize that for lateritic soils an amount of clay fraction larger than in podzolic soils would be allowable, for a certain plasticity index.

As the program of road construction in Brazil will expand toward the Central-West Region and as some strategical roads will probably be constructed in the North, it is certain that soil engineers will have to face increasingly the problem of using laterites as construction material.

We hope that basic research on clay minerals, development of engineering pedology, increase in soil mapping, and more closely supervised construction will build up a great amount of knowledge of soils of tropical regions.

It is also our belief that it would be highly profitable to establish some permanent exchange of information on soil engineering data from tropical countries. Perhaps some high-level coordination of such an interchange, by an international board under the auspices of the United Nations, should be endeavored.

F. L. D. WOOLTORTON.[4]—There is an aspect of lateritic soils which should be brought to the attention of engineers who must, by national policies, be becoming ever increasingly interested in the tropics or what is almost synonymous—the "Backward areas".

It used to be thought that lateritic soils, as opposed to laterite, were characterized by the presence of the inactive clay mineral Kaolinite. More recently it has been ascertained that the lateritic clay fraction may be characterized by one of a number of clay minerals. Attention to this, in the engineering field, was most strikingly drawn by Lambe and Martin in their series of papers to the Highway Research Board on "Composition and Engineering Properties of Soils", 1953-56. The pedological engineer must now wonder whether the Suborder of Lateritic Soils is sufficiently revealing to indicate the engineering properties he expects to be able to prognosticate from Agricultural Soil Classification.

There is a soil in Kenya which farmers sometimes refer to as "Red Cotton" soil. If that term means anything it suggests a red soil with the properties of a "Black Cotton" soil which, in engineering terms, means a soil having the properties of an active desiccated clay soil.

But farmers are knowledgeable of soils even though some of their expressions may be unscientific.

These "Red Cotton" soils could be lateritic soils containing some montmorillonite but they are more likely to be those whose clay fraction is composed almost wholly of one of the halloysite minerals which could be described as active forms of kaolinite.

The writer believes that halloysite is a far more commonly encountered clay mineral in the tropics than is so far acknowledged though he cannot recollect having seen lateritic soils formed under the monsoon conditions prevailing in Burma or India in which, with his present knowledge, he would say that halloysite was obviously present.

Lambe and Martin, quoting Grim, have pointed out that hydrated halloysite

3. "The Colloidal 'Activity' of Clays," Proceed. 3rd Intern. Conference Soil Mech. and Foundation Engg. (vol. 1).
4. Planning Engr., Ministry of Works, Nairobi, Kenya, B. E. Africa.

can change to metahalloysite at temperatures under 60° C. They have shown that hydrated halloysite can change to metahalloysite, in part, under the simple process of drying where some of this dehydration is irreversible. The interpretation to be attached to these observations is presumably that drying will tend to change the hydrated form into metahalloysite at temperatures below 60° C., where the lower the temperature the longer will be the drying time required for the change. In other words there is unlikely to be much change at lower temperatures when rain occurs every two or three days.

This is possibly illustrated by the following Kenya data:

1. Kinangop, altitude 8,500 feet, rainfall 50" occurring more or less uniformly throughout the year and recorded on about 170 days.
   Clay mineral* mainly hydrated halloysite with a small fraction of what is believed to be metahalloysite.
2. Kericho, altitude 6,500 feet, rainfall 70" occurring more or less uniformly throughout the year and recorded on about 170 days.
   Clay mineral* mainly metahalloysite with no evidence of hydrated halloysite.

It would appear from these two examples that the form in which halloysite occurs, for equal incidence of rainfall, seems to depend more upon temperature than on the amount of rainfall.

Lambe and Martin have commented very fully on the properties of the halloysite soils and there is little to add at this stage.

It is clear the soils must have a highly developed structure for although they may contain more than 50% voids they are not weak soils in their natural and undisturbed condition.

An observation possibly worthy of note is that although, in the areas where metahalloysite is found, there is little, if any, evidence of differential vertical movements as illustrated by cracks in light buildings constructed thereon, there is ample evidence of horizontal movement under bituminised road pavements in the crazing of these pavements which occurs. The difference between horizontal and vertical movements in some soils has been commented on by the writer elsewhere and may in this instance be a result of the orientation taken up by the tubular shaped mineral crystals.

S. F. GIZIENSKI,[5] M. ASCE.—Periodical engineering literature and engineering textbooks are extremely lacking in satisfactory outlines of the engineering properties of laterites and laterite soils. This shortcoming becomes most apparent to the practicing engineer when he is faced with the problem of making a decision on the feasibility of using these soils as construction materials.

The writer calls to mind a recent preliminary study of an earth dam project for which laterites found at a damsite in Brazil had to be considered for possible use in the pervious zone of the embankment. Clays of lateritic appearance and characteristics were considered for the impervious core of the dam. Because preliminary estimates had to be made before tests could be completed, it was necessary to formulate general opinions on the suitability of these materials for dam construction. A comprehensive search of the

* Supplied by Chief Materials Engineer, Ministry of Works, Kenya.

5. Principal Engr., Woodward, Clyde, Sherard and Associates, Oakland, Calif.

engineering literature led the writer to the same conclusion the author reached, that there is a scarcity of information on engineering properties of laterites and lateritic soils. The author's paper, which gives a comprehensive review of the basic laterite properties, from the engineering standpoint, is indeed very timely.

The long bibliographic list attached to the paper presents many interesting references. Only a few are of real engineering value, however, such as references 19, 48, 53 and especially reference 17. These supply fairly good information on laterites in French Equatorial Africa (Ref. 17), formed from dolerites, schists and granites, and on those formed from volcanic lava in Hawaii (Ref. 19).

Mr. Bawa has followed the classification of laterite materials established by the soil scientists, namely, (1) laterites, (2) lateritic soils and (3) non-lateritic soils. The criterion for this subdivision is the value of the silica-alumina ratio (replacing in this case the generally used silica-sesquioxide ratio). Obviously to apply this criterion it becomes necessary to make a chemical analysis of a given material. An engineer performing a routine soil investigation, especially one preliminary in character, usually is not in a position to do a chemical analysis. Actually a visual inspection in the majority of cases will suffice because materials described as laterites in the paper and found in the tropics are usually readily identified as laterites; doubt may exist insofar as lateritic soils only are concerned.

The engineer in the course of an investigation will have to treat the lateritic materials he encounters as he does other soils used in engineering structures. He should perform all tests usually performed on soils including those for swelling characteristics, taking into consideration the possible effect of reworking or manipulation. He will find that in spite of high plasticity (Plasticity Index up to 49, Ref. 17, p. 29) the laterite soils are non-expansive. In many cases they are pervious if tested in the natural condition and may be quite impervious if reworked. These soils may possess certain characteristic properties described by Fruhauf (Ref. 19) and others. According to Fruhauf some Hawaiian soil samples, if soaked in water for a long time, did not change their natural granular structure and behaved like a sand of considerable shearing resistance. If remolded, however, they became liquid, with a very high water content. Remolding could be done in some cases by simply using a spatula or a similar light effort, while in other cases a great amount of effort was required for remolding.

The engineer will try to classify the laterites and determine their Atterberg limits. In this connection the following information may be of value. Data by Willis (Ref. 19, p. 589) show that the results of the Atterberg tests depend very much on the preparation of samples. Air-dried field samples of Hawaiian soils did not show plasticity; but when the field samples as received were washed through a No. 40 sieve they showed Liquid Limits as high as 245 and Plasticity Indices as high as 110. These data tend to confirm Fruhauf's information.

The data by Florentin et. al. (Ref. 17) plotted on the Casagrande Plasticity Chart in the offices of the writer's firm gave the following results:

(a) Dolerite laterites. (4 samples) All samples fall below the A-line to the right of the B-line and should be classified as OH or MH. The average Atterberg values are Liquid Limit 67 and Plasticity Index 26.

(b) Granitic laterites. (13 samples) Three samples fall slightly above the

A-line, the rest being under it. Ten samples should be classified as OH or MH; the rest belong to the CH-group, but are not far from the OH or MH boundary. The average Liquid Limit is 68 and the Plasticity Index 28.

(c) Schistic laterites. (5 samples) All five samples fall below the A-line; two are to the left and three to the right of the B-line. Data referring to these groups are: the group of three samples are classified as OH or MH with average Liquid Limit of 64 and Plasticity Index of 27; the group of two samples is classified as ML or OL, with an average Liquid Limit of 44 and Plasticity Index of 14. Probably these two samples are lateritic soils, not laterites. Based on Florentin's data (discarding the two samples mentioned), the classification of all laterites and their Atterberg limits in French Equatorial Africa are approximately the same independent of the nature of their parent rocks.

Fruhauf also plotted the plasticity characteristics of the three Hawaiian lava lateritic soils (ref. 19, Table 2 and Fig. 2). All fall close to the A-line, and to the left of the B-line. The average values are Liquid Limit 41 and Plasticity Index 17. These figures are in agreement with the lower values of Table 5 of the paper; they are in disagreement with the general impression from Tables 4 and 5, that the lateritic soils are more plastic than the laterites.

Tests performed on lateritic soils for the previously mentioned dam project in Brazil show that the material has a Liquid Limit of 46 to 59 and a Plasticity Index of 15 to 29; the classification is close to CL-CH (medium clay). The percentage of sand sizes ranges from 12 to 29, the silt sizes from 16 to 22 and the clay sizes from 51 to 72. Maximum dry density in the Proctor tests was from 81 to 90 pounds per cubic foot and the optimum moisture content from 30 to 34 per cent. The average specific gravity of the particles was 2.68.

Engineering Use of Laterites

In addition to the author's opinions expressed under the heading "Engineering Behavior" the following may be added:

(1) Laterites generally have adequate strength to support ordinary engineering structures. Florentin et. al. (Ref. 17) call attention to the high compressive strength of the laterites in spite of their low dry density in the natural state.

(2) As construction materials laterites and lateritic soils may be used for earth dams and highways. In the case of earth dams the lateritic material may be used only if the following tests give satisfactory results: (a) effect of rolling on strength and permeability; (b) effect of prolonged saturation. The latter test may be dispensed with however, if the material in its natural state is permanently close to the state of full saturation. Florentin et. al. (Ref. 17) inform us, for example, that the average degree of saturation in the laterites of French Equatorial Africa is 90 per cent for doleritic laterites, 94 per cent for granitic laterites and 80 per cent for schistic laterites. The latter figure is relatively low, probably because of high lateral permeability due to schistosity of the material.

With respect to roads and highways it has been reported* that the presence of large lateritic fragments, in combination with other features, is the cause of corrugations similar to those observed on gravel roads.

* Highway Research Abstracts, vol. 28, January 1958, page 9.

GEOLOGIC INVESTIGATIONS OF DAM SITES BY THE SCS[a]

Discussion by John A. Trantina

JOHN A. TRANTINA,[1] M. ASCE.—The author has presented an excellent resume of the responsibilities of geologists associated with the dam building programs. Although the paper made reference to geological investigations of dam sites by the Soil Conservation Service, the general outline is applicable to the methods and procedures followed by state as well as by federal agencies.

Salaries

The author indicates that the Soil Conservation Service is purchasing drilling and sampling equipment, which will put the agency in competition with private enterprise. The discussion on costs, therefore, as a matter of interest, should not pass without comment. The author indicated that the total annual salaries of a 7-man drilling party is $23,420., or approximately $3345. per man. The party includes two geologists, a driller, a junior driller and three laborers. According to Civil Service Classification Standards an employee in charge of a field party, and assigned the technical responsibilities outlined in the paper, would have to be a GS-9 ($6250 per annum). His assistant would have to be a GS-7 ($5335 per annum). The wage scale of unclassified personnel in Civil Service is established in accordance with the local rates. It is difficult therefore to accurately define the drillers' salary. Assuming a minimum hourly rate of $2.25 for the Driller (which is low for a nationwide average) and $2.00 for the Junior Driller, their respective annual salaries would be $4680 and $4160. According to the above analyses, a balance of $3000 represents the total annual wage of the three laborers. If the above assumptions are in accord with the actual salaries paid to the classified and skilled personnel, then obviously .50 per hour for laborers is unreasonable and the $23,420 figure is in error. These estimates, furthermore, allow nothing for relief help for vacations or sickness or for overtime. If the assumed salaries of the geologists and drillers are lowered in order to raise the laborers salaries to a realistic level, then obviously the salary-responsibility ratio is out of line and the man-in-charge is underpaid.

Unit Costs

Inasmuch as the paper was presented in an informative manner to engineers who are not directly associated with subsurface explorations, the cost of such

---

a. Proc. Paper 1429, November, 1957, by Gunnar M. Brune.

1. Eng. Geologist, Woodward, Clyde, Sherard and Associates, Cons. Civ. Engrs., Oakland, Calif.

investigations are judged on a unit basis. The author indicated that the unit cost of drilling a site is $2.07 per foot. The unit cost figure does not include such items as insurance, office overhead and miscellaneous expenditures. Furthermore, several of the cost items listed appear unreasonably low. A depreciation rate of 12-1/2 per cent on vehicles and on drilling equipment is unrealistic, to put it mildly. Fuel and lubricants are indicated to run less than $10.00 per working day, and maintenance slightly more than $10.00 per working day, for five pieces of motorized equipment. It appears that the total annual cost should be increased by a very substantial percentage if realistic estimates of the cost of all items are included.

Drill Hole Location

After doing exploratory work with a government agency for many years, the writer feels that drill holes should not be located on a basis of 100- to 200-foot spacing, but rather on the basis of the geological pattern developed from a general surface survey. The preliminary report together with the geological reconnaissance survey should serve as a guide for tentatively locating the structures. Holes should then be drilled at strategic locations to confirm the foundation conditions.

This method does not necessarily conserve drilling but it does tend to develop a more comprehensive pattern of the subsurface conditions.

Drive Samples

The writer feels that drive samples should not be regarded as being unsatisfactory. Although the samples are disturbed, the method, when properly developed, is an excellent and economical means of sampling and defining the stratification of sands and gravels. When ground water makes sampling of sand and gravels rather difficult casing is frequently driven with drive sampling equipment. Although drive sampling is slower than sampling with spiral and bucket type augers, the equipment is regarded as being the most versatile in areas where variable conditions and materials are encountered.

CONCLUSION

Field explorations for dam sites are generally made in accordance with procedures and methods developed by experience and personal judgment. The comments contained in this discussion, therefore, were not intended to criticize the procedures outlined in this excellent paper.

GARRISON DAM—EVALUATION OF RESULTS FROM TUNNEL TESTS SECTION[a]

Discussion by A. A. Eremin

A. A. EREMIN,[1] A. M. ASCE.—In this paper and the preceeding one, the authors have described an interesting tunnel project, consisting of a series of eight parallel tubes, placed at short distances between centers. Methods of design and construction of the tunnels at the Garrison dam project were developed with the test tunnel bore, which later was made a part of the final series of tubes. This method of investigation of the ground by penetrating with the bore has been known for many years.

The test tunnel opens the ground and gives an unobstructed access to the ground for the exploring purposes. Disadvantages of this method are that it is costly and time consuming. This method of exploration has been used in the cases when the test tube could be used as a part of the final tunneling as it was the case at the Garrison tunnels. With similar intension many railroad tunnels were bored with timber supports, which were replaced with permanent concrete lining after the extensive study of the ground in the bore were performed. This method of ground exploring has also been used when the exploring tunnel could be used for the utilities as in the case of the Moffat railroad tunnel in Colorado, where the exploring pilot tunnel was later used for conveying of water in the waterwork project.

Mr. Lane has stated, ". . . a considerable saving in steel amounting to around three quarter of a million dollars . . . a much larger saving is believed to have accrued from the lower bids, resulting from the opportunity the test tunnel afforded bidders to appraise the work with much less uncertainity than usual in tunneling."

The well known methods of ground exploring with the drill test bores have given recently satisfactory informations for the contractors in estimating the cost of many tunnel projects.

The Pali tunnels for a two-lane highway bored through the Koolau mountain ridge near Honolulu, Hawaii, constructed in 1957, were designed after a careful study of the ground by the drill bores tests. Construction of the Pali tunnels was performed according the design plans, based on the bore-test information without any major changes and, besides, the construction was completed ahead of the time assumed for completion of the project. Furthermore, with minor changes, the same tunnel design details were adopted for construction of the new parallel tunnels, which are under construction at present.

a. Proc. Paper 1439, November, 1957, by K. S. Lane.

1. Assoc. Bridge Engr., California State Bridge Dept., Sacramento, Calif.

It is also interesting to note that the lowest bidder on new tunnels has shown almost the same unit prices for the tunnel excavation and tunnel concrete lining items with the usual corrections, according to the cost index factors, shown in Engineering News Record.

It is interesting to note, the lining of the Pali tunnels consists of the reinforced concrete arch with the embedded steel ribs similar to those used in the Garrison tunnels, except the Pali tunnels are, as generally termed, with a "horseshoe" section. The Pali tunnels are described by the writer in his recent publication.[2]

Expressing the ground load on the tunnel lining by the cylindrical section, Mr. Lane stated, "Vertical planes have been taken one diameter apart in view of evidence from lignite mines in the Fort Union, where collapse of a drift produces a depression at the surface, which is not wider than a drift."

The writer believes, Mr. Lane did not limit his conclusions to the specific ground conditions encountered at the Garrison dam tunnels.

In his conclusions Mr. Lane stated, "As with most investigations some of the results were reasonably definite, some inconclusive, and some unanticipated."

The author should be congratulated for presenting valuable information on the stress and strain measurements in the circular steel ribs and reinforced concrete lining. No doubt, this information will be useful in the further research of the tunnel supports.

2. Tunnels, Underground Structures, and Air Raid Shelters, by A. A. Eremin, p. 114, "Civil Engineer", March, 1958.

A METHOD TO DESCRIBE SOIL TEMPERATURE VARIATION[a]

Discussion by D. C. Pearce

D. C. PEARCE.[1]—The writer would like to draw to the attention of the authors of this interesting paper the results of recent ground-temperature studies of the Division of Building Research of the National Research Council of Canada. The work was reported at the 11th Annual Assembly of the International Union of Geodesy and Geophysics in Toronto, Ontario in September 1957 and will appear in the proceedings of that assembly.

The analysis was based on ground-temperature observations obtained at Ottawa, Ontario and Saskatoon, Saskatchewan, for the years 1952 to 1955. The annual temperature wave was determined by graphical harmonic analysis of the field data. Then the variation of the phase and amplitude of this wave with depth was studied. Results quite similar to those presented by the authors were obtained. In each case, the behaviour of the annual temperature was consistent with linear heat conduction theory. The parameter $\frac{P}{\pi \alpha}$ was relatively constant from year to year and ranged in value from 0.13 ft.$^{-1}$ for sand to 0.17 ft.$^{-1}$ for clay soil.

The description of the annual ground-temperature variations by conduction theory is interesting because heat transfer in a moist soil is not a pure conduction process. In an attempt to obtain additional information, a mathematical investigation of periodic heat flow in a medium with space-dependent thermal properties was undertaken. The conclusion was that the resulting temperature distribution would be very nearly equal to that in a homogeneous medium provided the distance over which the thermal constants are appreciably perturbed from the average value is small compared to the length of the temperature wave. The annual temperature variation apparently satisfies this condition. It is quite probable, however, that the condition would not be satisfied for the daily temperature variation so that the soil thermal properties obtained from a study of the daily wave might be quite different. In general it would appear that the thermal properties obtained may depend on the measurement technique, so caution should be exercised in using the resulting values for design purposes. This phenomenon is now under investigation at the Division of Building Research.

---

a. Proc. Paper 1537, February, 1958, by E. B. Penrod, W. W. Walton and D. V. Terrell.

1. Building Services Section, National Research Council of Canada, Ottawa, Canada.

CEMENT AND CLAY GROUTING OF FOUNDATIONS: GROUTING WITH CLAY-CEMENT GROUTS[a]

Discussion by Robert W. Cunny

ROBERT W. CUNNY,[1] J. M. ASCE.—Mr. Johnson suggests that a groutability ratio might be developed relating the properties of the foundation to those of the solid particles in the grout. During the period 1952 to 1954, the U. S. Army Engineer Waterways Experiment Station conducted a laboratory investigation to investigate the feasibility of grouting foundation sands with cement-type grouting agents. Results of this investigation were reported[2] to the Office, Chief of Engineers, in June 1955. The laboratory investigation, in part, was directed toward determination of the minimum grain size of sand that could be successfully grouted with the cement-type grouts being considered. Gradation curves of the grouting agents tested are shown in Fig. 1, and likewise, gradation curves of uniformly graded sand samples used for injection tests are shown in Fig. 2. The D_{85} size for the grouting agents tested and the D_{15} size for the sand samples are listed in the following table:

D_{85} Size for Grouting Agent and D_{15} Size for Sand Samples

Grouting Agent		Sand	
Type	D_{85} Size in mm.	No.	D_{15} Size in mm.
Scalped Type III Portland Cement (less than 30 microns)	0.017	3	0.72
Commercial Type III Portland Cement	0.026	4	0.62
Slag Cement	0.038	5a	0.37
Ground Slag	0.018	5	0.32
		6	0.20
		7	0.14
		8	0.06

The grouting equipment included an electrically-driven reciprocating pump of 536-in. cu in./min. displacement capable of delivering grout at pressures in excess of 100 psi, and a combination grouting chamber and permeameter,

a. Proc. Paper 1545, February, 1958, by S. J. Johnson.

1. Chief, Project Investigation Section, Embankment and Foundation Branch, Soils Div., U. S. Army Engr. Waterways Experiment Station, Corps of Engrs., Vicksburg, Miss.
2. U. S. Army Engineer Waterways Experiment Station, Corps of Engineers, T.M. 3-408, Grouting of Foundation Sands and Gravels. Vicksburg, Miss., June, 1955.

5 in. diameter by 9 in. high. The pump line from the grout pump to the permeameter was arranged in such a way that the grout specimen was permeated from the bottom upward.

Sands to be grouted in the permeameter were prepared for grouting in a manner similar to that for a permeability test, and at densities corresponding as nearly as possible to natural densities for natural sands. Prepared sands were placed with a moderate degree of compaction so that a relative density of approximately 70 to 80% would result. Specimens were prepared under water maintained at a height of approximately 1 in. above the surface of the specimen to minimize entrapped air; the material was placed using procedures developed[3] to avoid segregation of the sand grains. The specimens were approximately 6 in. long. A perforated plate, with 3/16-in. holes, was provided at the bottom and was covered by a 16-mesh screen followed by a 1/2-in. thickness of Ottawa sand which was used to support the specimen. A 16-mesh screen and perforated plate were placed at the top of the specimen and held down by a threaded rod to prevent uplift during grouting operations.

A series of tests were performed to determine the minimum size sand which could be successfully grouted with each of the grouting agents. Results of these tests are shown graphically in Fig. 3. Each bar represents the limits of groutability as determined from the laboratory investigations, based on the ratio of the D_{15} sand size to the D_{85} grout size. The right hand of each bar represents the sand-grout ratio for the finest sand proven groutable, while the left hand represents the sand-grout ratio for the coarsest sand proven not groutable. It may be noted that the cement-grouts tested did not penetrate a sand having a sand-grout ratio less than 18, but that each cement-grout tested penetrated material having a sand-grout ratio less than 24. As a result of these laboratory experiments, it appears that between the limits of 18 and 24 there probably exists a sand-grout ratio above which all sand-grout combinations are groutable.

3. Chen, Liang-Sheng, Stress-deformation and Strength Characteristics of Cohesionless Soils. Harvard Graduate School of Engineering, 1944.

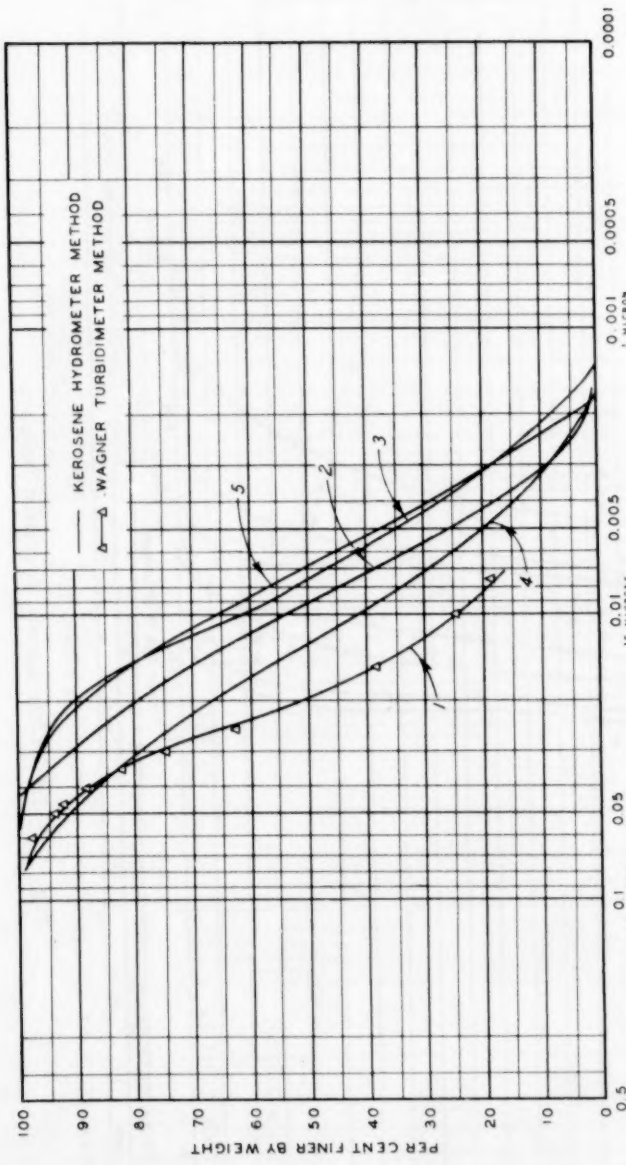


Fig. 1. Gradation curves for cement-type grouting agents

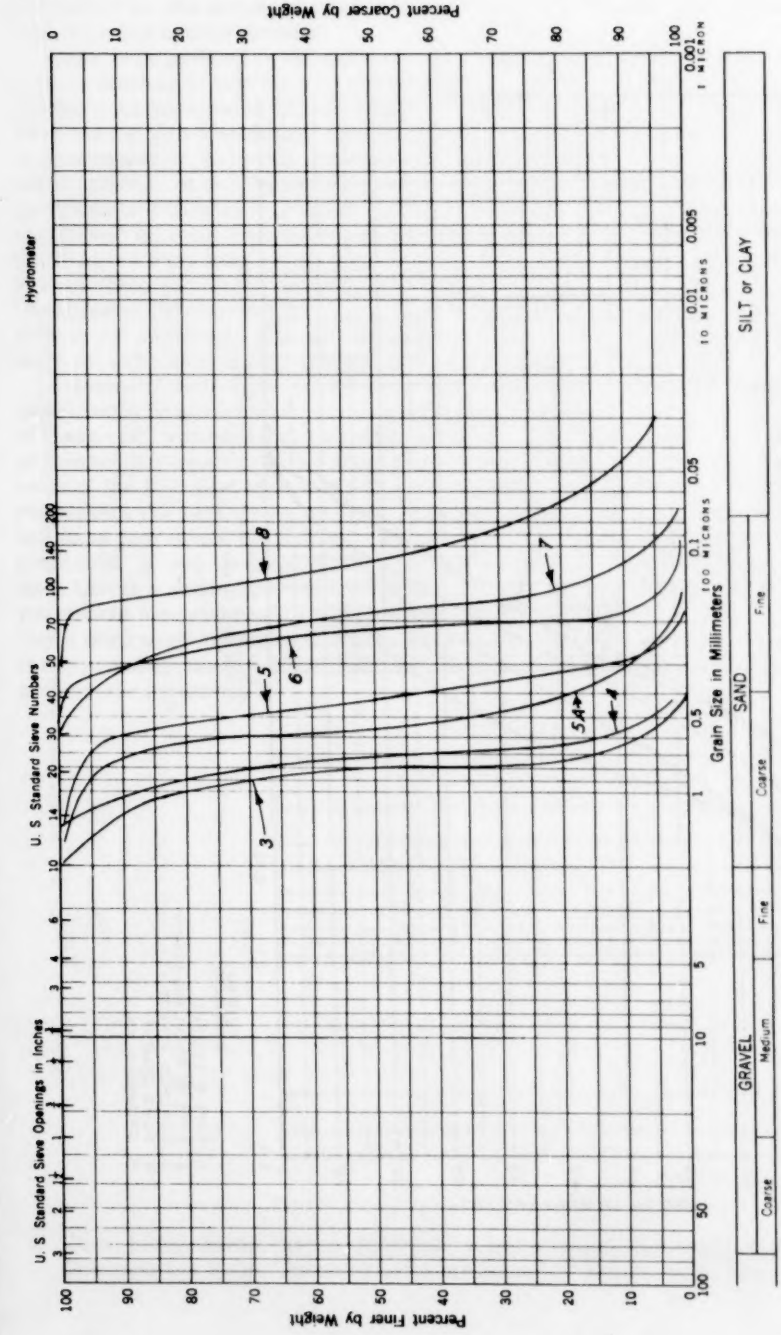


Fig. 2. Gradation curves for sand samples

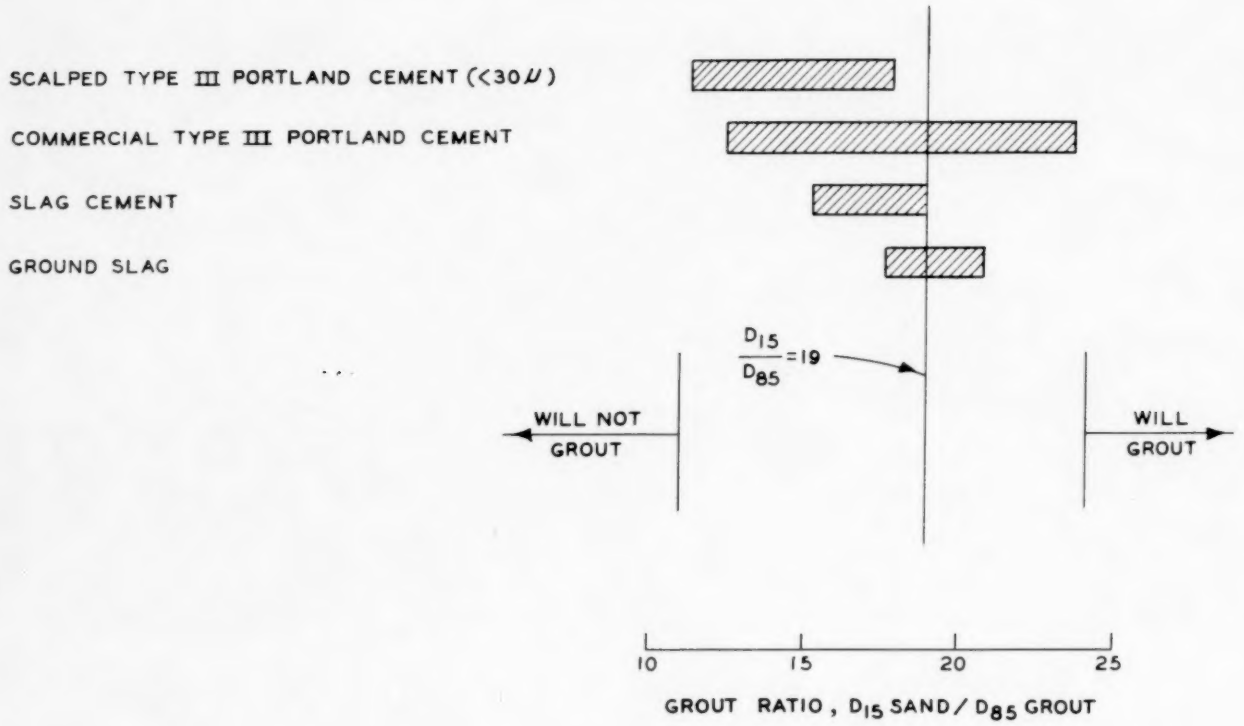


Fig. 3. Results of tests showing not groutable and groutable sand-grout combinations

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1449 is identified as 1449 (HY 6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1957.

VOLUME 83 (1957)

MAY: 1231(ST3), 1232(ST3), 1233(ST3), 1234(ST3), 1235(IR1), 1236(IR1), 1237(WW2), 1238(WW2), 1239(WW2), 1240(WW2), 1241(WW2), 1242(WW2), 1243(WW2), 1244(HW2), 1245(HW2), 1246(HW2), 1247(HW2), 1248 (WW2), 1249(HW2), 1250(HW2), 1251(WW2), 1252(WW2), 1253(IR1), 1254(ST3), 1255(ST3), 1256(HW2), 1257 (IR1)[c], 1258(HW2)[c], 1259(ST3)[c].

JUNE: 1260(HY3), 1261(HY3), 1262(HY3), 1263(HY3), 1264(HY3), 1265(HY3), 1266(HY3), 1267(PO3), 1268 (PO3), 1269(SA3), 1270(SA3), 1271(SA3), 1272(SA3), 1273(SA3), 1274(SA3), 1275(SA3), 1276(SA3), 1277 (HY3), 1278(HY3), 1279(PL2), 1280(PL2), 1281(PL2), 1282(SA3), 1283(HY3)[c], 1284(PO3), 1285(PO3), 1286 (PO3), 1287(PO3)[c], 1288(SA3)[c].

JULY: 1289(SM3), 1290(EM3), 1291(EM3), 1292(EM3), 1293(EM3), 1294(HW3), 1295(HW3), 1296(HW3), 1297 (HW3), 1298(HW3), 1299(SM3), 1300(SM3), 1301(SM3), 1302(ST4), 1303(ST4), 1304(ST4), 1305(SU1), 1306 (SU1), 1307(SU1), 1308(ST4), 1309(SM3), 1310(SU1)[c], 1311(EM3)[c], 1312(ST4), 1313(ST4), 1314(ST4), 1315 (ST4), 1316(ST4), 1317(ST4), 1318(ST4), 1319(SM3)[c], 1320(ST4), 1321(ST4), 1322(EM3), 1323(AT1), 1324 (AT1), 1325(AT1), 1326(AT1), 1327(AT1), 1328(AT1)[c], 1329(ST4)[c].

AUGUST: 1330(HY4), 1331(HY4), 1332(HY4), 1333(SA4), 1334(SA4), 1335(SA4), 1336(SA4), 1337(SA4), 1338 (SA4), 1339(CO1), 1340(CO1), 1341(CO1), 1342(CO1), 1343(CO1), 1344(PO4), 1345(HY4), 1346(PO4)[c], 1347 (BD1), 1348(HY4)[c], 1349(SA4)[c] 1350(PO4), 1351(PO4).

SEPTEMBER: 1352(IR2), 1353(ST5), 1354(ST5), 1355(ST5), 1356(ST5), 1357(ST5), 1358(ST5), 1359(IR2), 1360 (IR2), 1361(ST5), 1362(IR2), 1363(IR2), 1364(IR2), 1365(WW3), 1366(WW3), 1367(WW3), 1368(WW3), 1369 (WW3), 1370(WW3), 1371(HW4), 1372(HW4), 1373(HW4), 1374(HW4), 1375(PL3), 1376(PL3), 1377(IR2)[c],1378 (HW4)[c], 1379(IR2), 1380(HW4), 1381(WW3)[c], 1382(ST5)[c], 1383(PL3)[c], 1384(IR2), 1385(HW4), 1386(HW4).

OCTOBER: 1387(CP2), 1388(CP2), 1389(EM4), 1390(EM4),1391(HY5), 1392(HY5), 1393(HY5), 1394(HY5), 1395 (HY5), 1396(PO5), 1397(PO5), 1398(PO5), 1399(EM4), 1400(SA5), 1401(HY5), 1402(HY5), 1403(HY5), 1404 (HY5), 1405(HY5), 1406(HY5), 1407(SA5), 1408(SA5), 1409(SA5), 1410(SA5), 1411(SA5), 1412(EM4), 1413 (EM4), 1414(PO5), 1415(EM4)[c], 1416(PO5)[c], 1417(HY5)[c], 1418(EM4), 1419(PO5), 1420(PO5), 1421(PO5), 1422(SA5)[c], 1423(SA5), 1424(EM4), 1425(CP2).

NOVEMBER: 1426(SM4), 1427(SM4), 1428(SM4), 1429(SM4), 1430(SM4)[c]. 1431(ST6), 1432(ST6), 1433(ST6), 1434(ST6), 1435(ST6), 1436(ST6), 1437(ST6), 1438(SM4), 1439(SM4), 1440(ST6), 1441(ST6), 1442(ST6)[c], 1443(SU2), 1444(SU2), 1445(SU2), 1446(SU2), 1447(SU2), 1448(SU2)[c].

DECEMBER: 1449(HY6), 1450(HY6), 1451(HY6), 1452(HY6), 1453(HY6), 1454(HY6), 1455(HY6), 1456(HY6)[c], 1457(PO6), 1458(PO6), 1459(PO6), 1460(PO6)[c], 1461(SA6), 1462(SA6), 1463(SA6), 1464(SA6), 1465(SA6), 1466(SA6)[c], 1467(AT2), 1468(AT2), 1469(AT2), 1470(AT2), 1471(AT2), 1472(AT2), 1473(AT2), 1474(AT2), 1475(AT2), 1476(AT2), 1477(AT2), 1478(AT2), 1479(AT2), 1480(AT2), 1481(AT2), 1482(AT2), 1483(AT2), 1484(AT2), 1485(AT2)[c], 1486(BD2), 1487(BD2), 1488(PO6), 1489(PO6), 1490(BD2), 1491(BD2), 1492(HY6), 1493(BD2).

VOLUME 84 (1958)

JANUARY: 1494(EM1), 1495(EM1), 1496(EM1), 1497(IR1), 1498(IR1), 1499(IR1), 1500(IR1), 1501(IR1), 1502 (IR1), 1503(IR1), 1504(IR1), 1505(IR1), 1506(IR1), 1507(IR1), 1508(ST1), 1509(ST1), 1510(ST1), 1511(ST1), 1512(ST1), 1513(WW1), 1514(WW1), 1515(WW1), 1516(WW1), 1517(WW1), 1518(WW1), 1519(ST1), 1520 (EM1)[c], 1521(IR1)[c], 1522(ST1)[c], 1523(WW1)[c], 1524(HW1), 1525(HW1), 1526(HW1)[c], 1527(HW1).

FEBRUARY: 1528(HY1), 1529(PO1), 1530(HY1), 1531(HY1), 1532(HY1), 1533(SA1), 1534(SA1), 1535(SM1), 1536(SM1), 1537(SM1), 1538(PO1)[c], 1539(SA1), 1540(SA1), 1541(SA1), 1542(SA1), 1543(SA1), 1544(SM1), 1545(SM1), 1546(SM1), 1547(SM1), 1548(SM1), 1549(SM1), 1550(SM1), 1551(SM1), 1552(SM1), 1553(PO1), 1554(PO1), 1555(PO1), 1556(PO1), 1557(SA1)[c], 1558(HY1)[c], 1559(SM1)[c].

MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2), 1568 (WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(ST2)[c], 1577(PL1), 1578(PL1)[c], 1579(WW2)[c].

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588 (HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)[c],1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)[c], 1615(IR2)[c], 1616 (HY2)[c], 1617(SU1), 1618(PO2)[c], 1619(EM2)[c], 1620(CP1).

MAY: 1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 (ST3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 1639(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647 (SM2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 1656(ST3)[c], 1657(SM2)[c].

---

c. Discussion of several papers, grouped by divisions.

AMERICAN SOCIETY OF CIVIL ENGINEERS

OFFICERS FOR 1958

PRESIDENT

LOUIS R. HOWSON

VICE-PRESIDENTS

Term expires October, 1958:
FRANCIS S. FRIEL
NORMAN R. MOORE

Term expires October, 1959:
WALDO G. BOWMAN
SAMUEL B. MORRIS

DIRECTORS

Term expires October, 1958:
JOHN P. RILEY
CAREY H. BROWN
MASON C. PRICHARD
ROBERT H. SHERLOCK
R. ROBINSON ROWE
LOUIS E. RYDELL
CLARENCE L. ECKEL

Term expires October, 1959:
CLINTON D. HANOVER, Jr.
E. LELAND DURKEE
HOWARD F. PECKWORTH
FINLEY B. LAVERTY
WILLIAM J. HEDLEY
RANDLE B. ALEXANDER

Term expires October, 1960:
PHILIP C. RUTLEDGE
WESTON S. EVANS
TILTON E. SHELBURNE
CRAIG P. HAZELET
DONALD H. MATTERN
JOHN E. RINNE

PAST PRESIDENTS

Members of the Board

ENOCH R. NEEDLES
MASON G. LOCKWOOD

EXECUTIVE SECRETARY
WILLIAM H. WISELY

TREASURER
CHARLES E. TROUT

ASSISTANT SECRETARY
E. LAWRENCE CHANDLER

ASSISTANT TREASURER
CARLTON S. PROCTOR

PROCEEDINGS OF THE SOCIETY

HAROLD T. LARSEN
Manager of Technical Publications

PAUL A. PARISI
Editor of Technical Publications

FRANCIS J. SCHNELLER, JR.
Assistant Editor of Technical Publications

COMMITTEE ON PUBLICATIONS

HOWARD F. PECKWORTH, *Chairman*

PHILIP C. RUTLEDGE, *Vice-Chairman*

E. LELAND DURKEE
R. ROBINSON ROWE
TILTON E. SHELBURNE
LOUIS E. RYDELL